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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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June, 1958

Papers

Journal of the

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Proceedings of the American Society of Civil Engineers

UNDERGROUND POWER PLANTS IN CANADA[a]

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(Proc. Paper 1670)

ABSTRACT

Design features of underground power plants vary considerably from country to country depending on the climate, topography and economics of development.

This paper presents a review of factors affecting design practice in Canada and fully describes two large underground power houses at present under construction in the Province of Quebec. Reference is also made to the Kemano development in British Columbia.

INTRODUCTION

Within the past six years construction of three underground hydro-electric stations has been commenced in Canada. Two of these, Kemano in Northern British Columbia, and Bersimis No. 1 in North-Central Quebec, are in successful operation. In both cases a planned partial development has been completed. The ultimate installation at Bersimis No. 1 is scheduled for completion in 1958. Construction of the third underground station, Chute-des-Passes, also in North-Central Quebec, was begun in 1956.

All three of these stations are large. At Kemano, the installed capacity is 1,120,000 horsepower and the gross head 2,600 feet. Plans have been made to double the capacity of this plant ultimately. The full planned capacity for Bersimis No. 1 is 1,200,000 horsepower and the gross head 875 feet. Chute-des-Passes with a smaller gross head varying between 540 and 640 feet will have an installed capacity of 1,000,000 horsepower in 1960.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1670 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 3, June, 1958.

a. ASCE, Convention, New York, Oct. 17, 1957.

1. H.G. Acres & Co., Ltd., Niagara Falls, Canada.

Prior to these installations, no power plants had been constructed in Canada which could properly be termed underground developments, the only approach to underground construction being the Toronto Power Company's station now owned the operated by the Hydro-Electric Power Commission of Ontario, and the Canadian Niagara Power Company's station, both at Niagara Falls, Ontario.(1) Both of these stations were built in the first decade of the present century. A section through the power house of the former development is shown in Figure 1. Here the turbines and shafts only are in underground galleries or pits, and both developments incorporate fairly long tailrace tunnels. Long shafts extend from the turbines to generators at ground level.

No shelf of rock was available in the gorge on which to construct a surface power house, so the logical arrangement was to locate the turbines underground. By locating the power house at the intake, operating forces were concentrated at one point. The layout permits the utilization of the shortest possible distance from the head of the rapids above the Falls to the Maid of the Mist pool at their foot.

It is impossible now to determine, with certainty, the various considerations which led the consulting engineers of the two developments to select the particular arrangement of turbine and generator which they did. Undoubtedly they realized that the seams in the limestone rock would be water bearing and, while a wet-walled pit would present no great problems as the location for water turbines, it was, no doubt, felt that the generators, generator leads, switching equipment, etc., and the turbine actuators should be located in a dry room above ground level, where this equipment would be close to the control room, offices, and operating staff. The course of events seems, anyway, to have fully justified the solutions which were devised to solve the problems encountered.

For over forty years thereafter, while Canada was busily developing 16,500,000 horsepower from her rich resources, no sites presented themselves at which the topography was suitable for underground power houses. This was undoubtedly due to the fact that these developments were all in the low to medium head range. This is typical of Canadian water power, where very few sites have heads greater than 500 feet, and it seems reasonable to believe that the higher the head the greater is the chance that the topography will either by favourable to, or demand, a layout comprising a typical underground installation.

This same period saw great advances in the art of underground rock excavation. Equipment was considerably improved and by 1945 drill steels with tungsten carbide tips became available and were widely adopted so that the speed of drilling was greatly increased. Better blasting powders and more efficient techniques for detonation and firing were devised. The impetus for much of this development came from the hard rock mining industry of Northern Ontario and Quebec, which concurrently went through a period of great expansion. Despite the universal marked increase in costs which has taken place during the period in question, it can be stated that, due to the events outlined, the costs of rock excavation have risen less than those of most other items of work involved in the construction of a hydro-electric power plant.

The conventional development involving a concrete dam and a surface power house constructed integrally with the dam, or on a bench on one of the abutments, is often not economically feasible at remote northern sites because

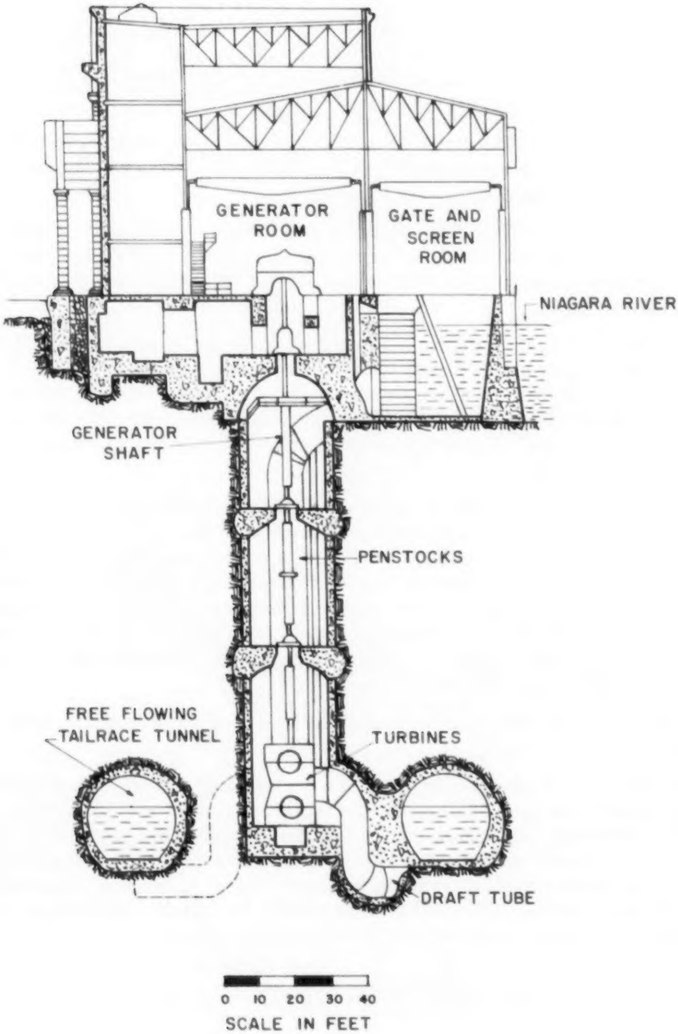

FIG. I TORONTO POWER GENERATING STATION

of the prohibitively high cost of transporting large quantities of cement, reinforcing steel and penstock steel.

The same sites can often be developed at lower cost by constructing earth or rock-fill dams to heights as great as 600 feet, and locating the power house and water conduits in sound rock in the abutments.

The principal advantages of underground power developments in Canada can be summarized as follows:

1) Since the layout of an underground development is not affected by topography to the same extent as a surface development, the most efficient arrangement can be chosen.

2) Winter construction difficulties are reduced to a minimum. Above-freezing temperatures which prevail underground eliminate most of the problems associated with outdoor construction in extremely cold winter months.

3) Underground developments are unaffected by the hazards of northern construction, namely, forest fires in the summer months, ice and snow in the winter months, and snow or rock slides on exposed cliffs, particularly during the spring thaw. In addition, costly insulation and heating of surge tanks, exposed penstocks and other waterways are reduced to a minimum.

4) The tonnage of steel and cement which must be transported is very considerably reduced and, consequently, when the site is located at a great distance from railhead or wharf, a substantial reduction of the over-all cost of development can be realized. In sound Pre-Cambrian rock of the type found in most parts of Northern Canada, particularly Ontario and Quebec, the tonnage of steel required for penstocks, surge tanks, etc. is much less for an underground development than for the corresponding surface development.

5) Maintenance of steel and concrete exposed to the elements is minimized.

A brief description of the Bersimis No. 1 and Chute-des-Passes underground developments is presented in this paper, together with an outline of the special problems associated with them, and the methods adopted in solving the problems encountered. A number of excellent papers have already been published describing the various features of the Aluminum Company of Canada's Kemano development,(8) so that except by way of illustration or comparison, this installation will not be discussed herein.

Types of Development

The type of underground power installation which will prove most appropriate to a particular site depends upon the head to be developed and the topography. Figure 2 shows the four principal types of underground development.(2)

Type A is most commonly used for high head schemes. The supply tunnel runs under low pressure and is safeguarded against water hammer by a surge tank. The major drop in elevation down to the power house is usually made by inclined or vertical penstocks driven in rock. The tailrace for the power

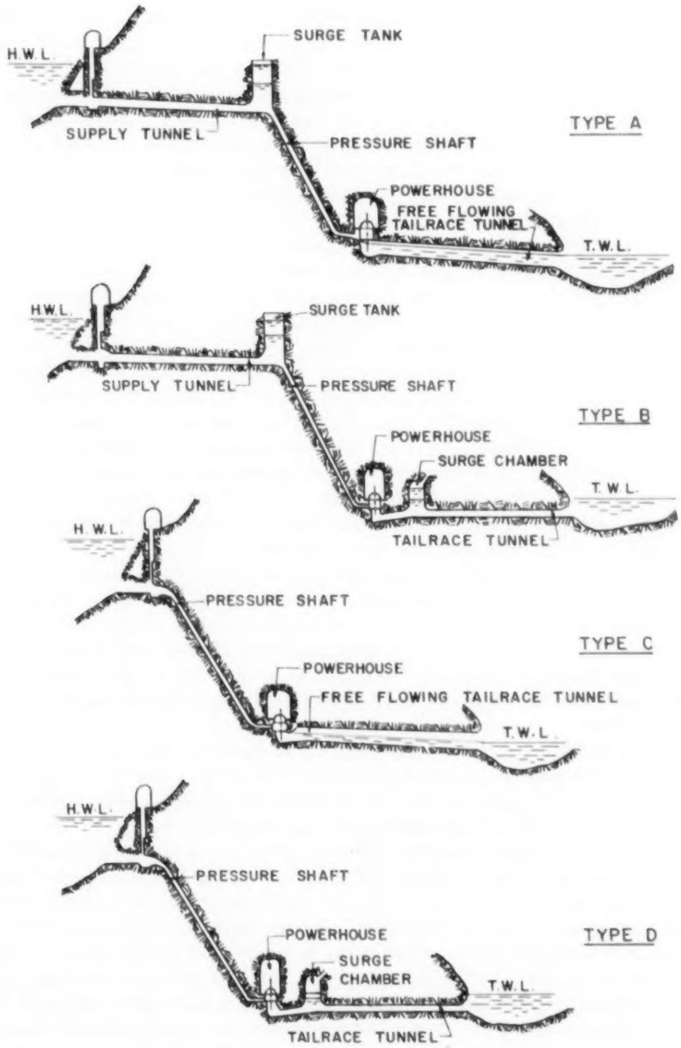

FIG. 2 TYPICAL SCHEMES OF UNDERGROUND DEVELOPMENT

house takes the form of a free flowing tunnel.

This method of development has been used in Canada both at the Kemano - Kitimat project and the Bersimis No. 1 project. A plan and profile of the latter development are shown on Figure 3. In both cases the adoption of a design incorporating an underground power house allowed the penstocks to be driven in rock - a feature which permitted a considerable reduction in the weight of steel required for the penstock liners and the realization of substantial overall economies. A circumstance which militated against the adoption of a surface power house for the Bersimis No. 1 development was the difficulty of providing suitable foundations for such a structure. Of all the methods of underground development, Type A provides the easiest access to the power house and gives the greatest safety against flooding.

The layout designated as Type B, the principal feature of which is a tailrace in the form of a pressure tunnel, has been used for the 1,000,000-horsepower Chute-des-Passes development in Quebec, illustrated in profile in Figure 4. Type B is an extension of Type A which is generally found to be economic when the length of tailrace tunnel exceeds 1,000 feet, or when the tailrace discharges into a body of water whose level fluctuates considerably. The design of the surge tanks for such a development involves many complex problems of stability[3,4] and for long tailrace tunnels the volume of the tailrace surge tank required to accommodate complete load rejection may become very great. Unless the turbines are set at a very low elevation relative to the tailwater, a tailrace tunnel weir may be required to prevent air from entering the draft tubes during down surges.

Types C and D, where the underground power house is situated directly below the intake, are primarily suitable for heads up to 300 feet. Access to the power house is gained by a vertical or inclined shaft. Type C is usually employed when the tailrace tunnel is comparatively short, and Type D where the length of the tailrace tunnel exceeds about 1,000 feet, or the tailwater fluctuations increase excessively the height of tailrace tunnel required to preserve free flow.

Types C and D have been used extensively in Sweden, where heads of between 100 and 300 feet are commonly available at sites suitable for underground development. In Canada, the early Niagara developments mentioned in the introduction could be classified as being Type C.

Neither of the two types, C or D, however, lend themselves to a quick construction schedule. For example, in the case of the Bersimis No. 2 development in Quebec, where the topography was favourable to underground development by either of these methods, a layout incorporating a surface power house was adopted since work on the power house could proceed independently of work on the penstocks and tunnel. With a scheme such as Type C, however, approximately one additional year would have to be allowed for power house excavation before it would be possible to place concrete for the turbine draft tubes and around the scroll cases. In this development, as in many others with short tunnels, the time for power house construction governs the completion date. Figure 5 shows the layout which was adopted to permit work on the tunnels and dams to proceed simultaneously with that in the power house. The topography adjacent to the power house permitted the penstock steel liners to be embedded in rock, thus retaining the advantages of this arrangement which are normally only available in an underground layout.

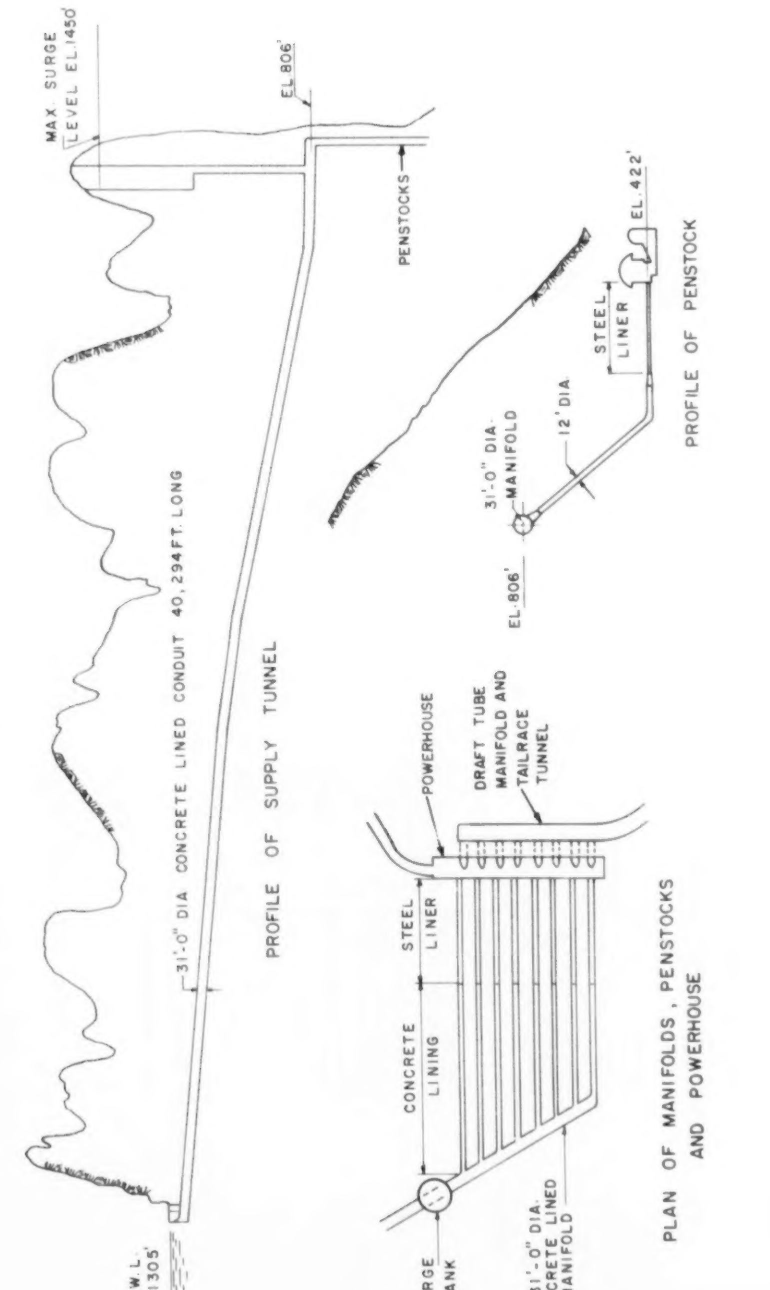

FIG. 3 PLAN AND PROFILES OF BERSIMIS NO. 1 DEVELOPMENT

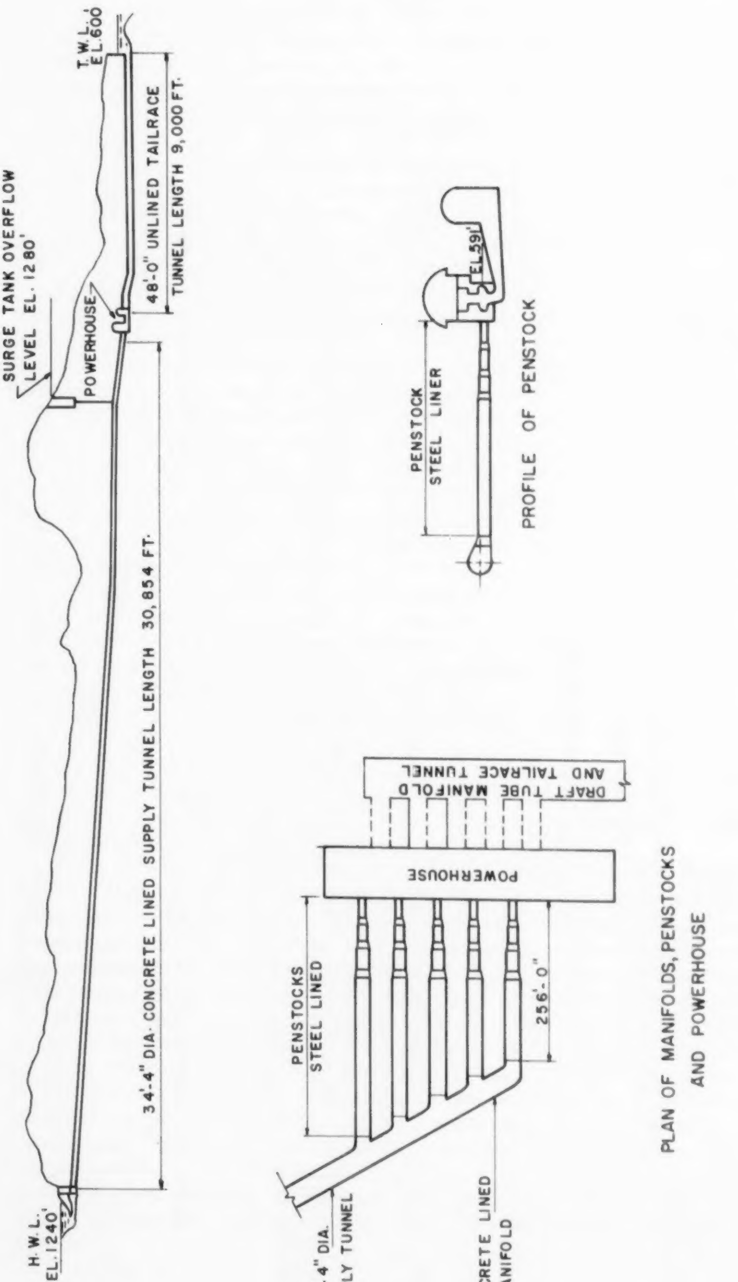

FIG. 4 PLAN AND PROFILES OF CHUTE-DES-PASSES DEVELOPMENT

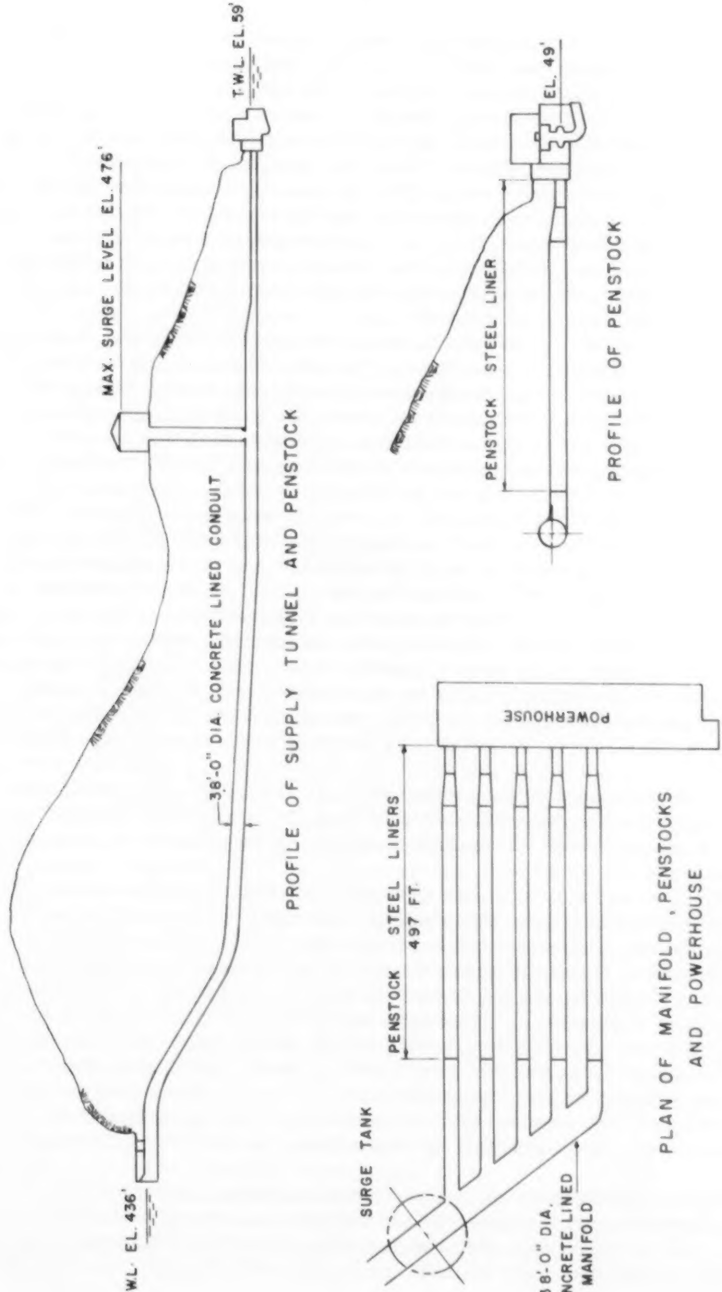

FIG. 5 PLAN AND PROFILES OF BERSIMIS NO. 2 DEVELOPMENT

Design Considerations

While the previous section has dealt with the general arrangement of underground developments as affected by topographic and hydraulic considerations, other factors have a profound influence.

In the Province of Quebec and in Northern Canada where the rocks mainly comprise the granites, gneisses, and paragneisses of the Canadian Shield, the stable geological conditions greatly favour the adoption of underground layouts. On the west coast, however, in the geologically complex Cordilleran region, the severe distortion of the rocks, and the consequent extensive faulting, demands that the greatest care be exercised in assessing the merits or demerits of an underground development. In any location it is essential that a thorough and comprehensive program of geological explorations be executed before a final layout is adopted.

Generally speaking, as the largest rock span and the most difficult part of the excavation is within the power house, the power house should be oriented in that direction which utilizes to the greatest extent the rock's arching action. This occurs when the strike of foliations and bedding planes is perpendicular to the power house axis. Furthermore, if the power house is so located, there is a tendency to ensure that inferior rock is confined to the shortest dimension of the power house. Where, as is often the case in developments of Types B, C, and D, the power house is at a considerable depth below the rock surface, care must be taken to verify that the rock is competent to sustain the increased crushing stresses induced by the excavation. If necessary, photoelastic studies should be made to assess the magnitude of these stresses.(5) It is important to investigate the condition of the rock around the penstocks. These normally enter the power house at right angles, and if the power house is so located that the strike of the rock foliations is perpendicular to its axis, the strike is necessarily parallel to the penstock axes. In these circumstances it is to be expected that overbreak will exceed that which would have occurred had the penstocks been driven normal to the strike.

The penstocks should be steel lined wherever the weight of the rock cover does not impose sufficient prestress to prevent the development of tensile stresses in the rock under the maximum surge and water hammer pressures which can occur.(6)

The layout of hydraulic conduits adjacent to an underground power house must be carefully considered if tunnelling costs are to be kept to a minimum. In underground developments where a single supply tunnel conveys water from the intake, it is usually most economical to terminate this tunnel in a manifold, the penstocks of a multi-unit station forming branches from this manifold. Recent hydraulic studies have indicated ways in which manifold and junction losses can be minimized(7) and for the Bersimis and Chute-des-Passes developments an extensive series of air model tests led to the evolution of very efficient branch transitions from a manifold of constant diameter. The use of a constant manifold diameter reduces tunnelling costs and the standard transition from manifold to penstock minimizes the cost of formwork.

To reduce water hammer pressures in the penstocks, the surge tank should be located as near as possible to the upstream end of the manifold. However, even if this is done the pressure rise, due to water hammer, may be high and calculations must be made, preferably with the aid of an electronic

computer, to evaluate its magnitude.

As mentioned earlier, a large tailrace tunnel surge tank is required to minimize the drop in water level at the draft tube outlets upon load rejection. For multi-unit stations this surge tank may also constitute a draft tube manifold in which draft tube gates may be located and handled.

An underground power development presents certain problems associated with the electrical installation. In such developments, the distance between the generators and the high-voltage switching station is usually much greater than in the case of surface power developments. This means that power must be transmitted from the generators to the switching station by means of long low-voltage leads, or by high-voltage cables, the choice depending upon whether the step-up transformers are above or below ground level.

Wherever possible, it is preferable to locate the step-up transformers at ground level to avoid the hazard associated with large volumes of transformer oil located within or directly adjacent to the power house. This arrangement, of course, results in rather long, low tension leads. Where, as in the Bersimis No. 1 and Chute-des-Passes developments, these leads are 400 to 500 feet long, studies have shown that long low-voltage leads are the economic solution when the cost of excavation for underground transformers is taken into account. For the large units usually associated with underground power developments (upwards of 100 megawatts) it will normally be found economical to excavate separate tunnels or shafts for the leads from each generator. These shafts, which are also for ventilation, can be rendered inaccessible while the leads are alive. Consequently, bare leads mounted on insulators can be used. For the capacities involved (over 5,000 amperes) these leads may take the form of aluminum channels. In vertical shafts the supports for these channels may also support access ladders and platforms required for maintenance work.

The control room with attached offices may adjoin the power house or may be a separate building above ground, usually close to the transformers and the high-voltage switching yard. The choice is usually a matter of the owner's preference since the cost difference is generally small.

Where the site is remote, it is not feasible to ship transformers to the manufacturer for repairs, and in such cases it is usual to arrange for the transportation of the main transformers to the power house where the power house crane may be used to facilitate repairs, thus saving the cost of a separate transformer repair building.

The ventilation requirements for underground and above ground power houses differ considerably. Underground power houses require relatively greater quantities of ventilating air, particularly in summer, since all heat not removed by the generator air to water heat exchangers must be removed by the ventilating air. The air is usually brought in through the main adit to the power house by fans, and secondary fans are used when necessary to ensure a through air circulation. The air is then exhausted through the generator lead shafts where it also serves to remove the quite considerable quantity of heat generated by the heavy current in the leads. In cold weather, the generators are used as a source of heat, a proportion of their cooling air being circulated through the power house.

A problem which may be encountered when the temperature rises after a period of cold weather is icing in the air intake. It may be necessary to supply heat at this point to prevent such troubles.

A reasonable intensity of illumination on the generator room floor and ceiling is essential. Usually the ceiling is illuminated by some form of floodlight placed above and behind the crane rails. Generally direct lighting of the floor is provided, using fluorescent fixtures, the arrangement of which depends largely on whether or not a false ceiling is incorporated under the concrete power house roof.

Description of the Three Major Canadian Underground Power Developments

(a) - Kemano

The design and construction of the Kemano development have been very adequately described in a number of papers published during the last few years. Several excellent papers fully describing this development are listed in the bibliography.[8]

(b) - Bersimis No. 1 Development[9]

The Bersimis No. 1 development is located on the Bersimis River which flows into the St. Lawrence on the north shore approximately 190 miles downstream from Quebec City. The development supplies electrical power to the Quebec Hydro-Electric Commission's distribution system for consumption in the Quebec and Montreal area, and to a lesser extent on the south shore of the St. Lawrence River and the Gaspe Peninsula. The first 4 of the 8 units to be ultimately installed were placed in successful operation early in 1957.

The Bersimis River falls approximately 1,225 feet in the 100 miles between Lake Pipmaucan and the mouth of the river. Of this drop, approximately 720 feet is concentrated in a series of falls and rapids over a stretch of approximately 20 miles below Lac Casse. The Bersimis No. 1 development harnesses the power potential associated with this difference in elevation.

The drainage area upstream from the outlet of Lac Casse is 5,010 square miles and the reservoir area 290 square miles. Studies indicated that a regulated mean flow of 9,250 cfs could be maintained at the outlet of Lac Casse and that 98 per cent of the average flow in the river could be utilized by forming a reservoir with a live storage capacity of 168 billion cubic feet.

From a series of studies the most suitable scheme of development was evolved. This scheme consists of raising the water level in Lac Casse approximately 200 feet, by the construction of two rock-fill dams at the output of Lac Casse, passing water through a power tunnel from an intake situated at the end of an arm of the reservoir, for a distance of 7-1/2 miles to a manifold, and thence through individual penstocks to turbines located in an underground power house near the Bersimis River. Profiles of the tunnel and penstocks and a plan and section of the power house are shown in Figure 3.

The intake consists of eight 16-foot wide by 30-foot high openings which direct the water into the power tunnel. After exhaustive model tests,[10] an intake combining economy with minimum possible head loss was designed. Immediately upstream from a transition section leading into the power tunnel, an intake head gate for a 30-foot by 30-foot clear opening is provided.

The gate is normally raised and dogged above deck level and is lowered only for unwatering of the power tunnel. No emergency tripping is provided since the possibility of a sudden failure of the power tunnel or penstocks is considered to be very remote, and the turbines may be isolated from the penstocks by means of penstock valves located immediately upstream from the units.

The power tunnel begins at the intake with an invert elevation of 1,235 feet and terminates at the penstock manifold with an invert elevation of 791 feet, dropping 444 feet in the 7-1/2-mile length of the tunnel. Studies showed that it was economic to provide the tunnel with a concrete lining and that the optimum internal diameter of the circular tunnel was 31 feet. For construction reasons an equivalent horseshoe section was adopted. The concrete tunnel lining was not reinforced. Any void remaining between the concrete and the rock at the crown of the tunnel was filled by pressure grouting.

Excavation in the tunnel was performed by three contractors, operating in three different sections simultaneously. Access was provided through adits located approximately midway in each section. The adits were of roughly rectangular cross section with a width of 32 feet and a height of 25 feet, the roof being arched sufficiently to provide safe working conditions during construction. The total adit length was 6,600 feet and the maximum grade was 8.9 per cent.

The tunnel was driven full size with excavation progressing at six faces simultaneously. Approximately 1,678,000 cubic yards of rock were excavated from 46,600 feet of tunnel and adit. Excavation was completed in 18 months, work continuing 24 hours a day, 6 days a week, throughout the entire period. The tunnel concrete arch was completed in advance of the invert to reduce cleanup on the invert at the end of the job.

Immediately upstream from the penstock manifold, a restricted orifice surge tank is provided to limit pressure rises in the tunnel and penstocks. This tank is located entirely in rock and was not lined since the rock proved to be very sound. A rock trap is, however, provided at the bottom of the tank to prevent rock which may be dislodged from the walls from falling into the tunnel.

The power tunnel terminates in a horizontal manifold, 31 feet in diameter, from which eight 12-foot diameter concrete-lined penstocks take off at an angle and lead to the eight turbines in the power house. The penstocks drop at an angle of 50 degrees in the horizontal from elevation 791 feet at the manifold to vertical elbows at elevation 422 feet, and then run approximately horizontally to the power house. For the last 328 feet the penstocks are lined with steel and their diameters are decreased in stages from 10'-0" to 7'-9". The 2-foot lining of the concrete penstocks and the 2-foot backing between the steel lining and the rock were placed using pumpcrete machines. Pressure grouting in stages up to a maximum pressure of 300 psi and to a depth of 15 feet in rock was specified in order to fill any possible voids and to provide a strong and relatively watertight shell around the penstock.

During excavation of the raises in penstocks Nos. 1 and 2 it was found to be expedient to excavate a cut across penstocks Nos. 1, 2, and 3 just downstream from the elbows. This gave access to the sloping portions of Nos. 1 and 2 from penstock No. 3 and permitted excavation to be continued in the power house for units Nos. 1 and 2 below the penstock level.

The power house chamber is 565 feet long, 65 feet wide and 80 feet high.

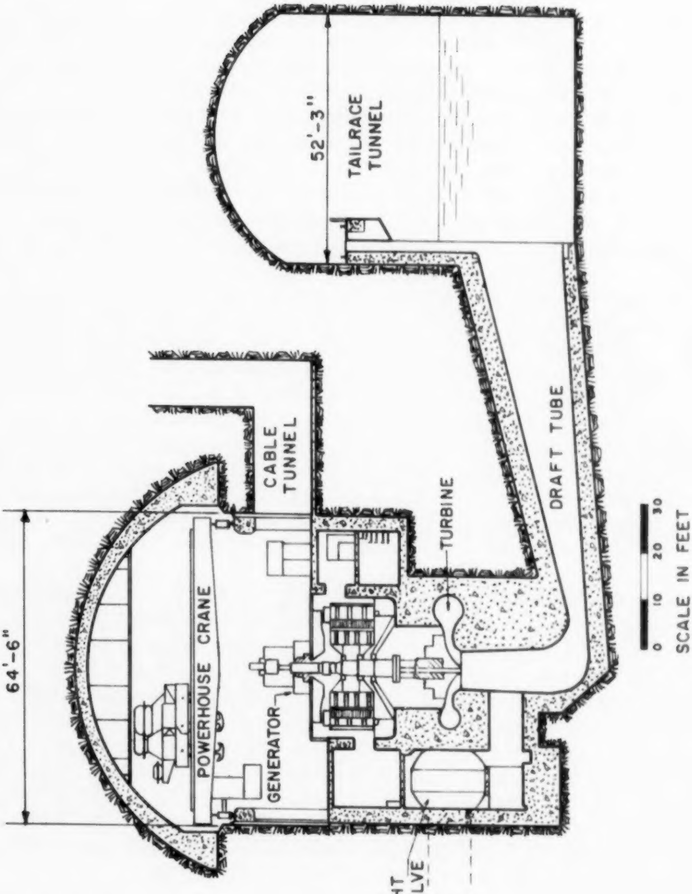

FIG. 6 CROSS SECTION OF BERSIMIS No. 1 POWERHOUSE AND TAILRACE TUNNEL

The turbine draft tubes discharge into an underground tailrace chamber 47 feet wide by 75 feet high, communicating with a short tailrace tunnel which, in turn, carries the water into the tailrace channel leading to the Bersimis River. Two sets of draft tube gates are provided to permit unwatering of the draft tubes.

Excavation of the power house and tailrace chamber was carried out in the sequence shown in Figure 7. The concrete arch in the power house was placed as soon as the progress of excavation would permit, and is Stage 4 in the sequence of operations. As excavation of the power house roof progressed, rock bolts were installed where necessary, to support the roof arch rock. This method of rock stabilization, which for a number of years has been used extensively in American and Canadian mines, is finding increasing application in civil engineering rock excavation work in Canada. Drilling and blasting of the rock designated as Stage 3 was completed before the concrete roof arch was placed. The blasted rock was levelled and left in place to act as a foundation for the timber framework of the arch forms. As soon as the concreting operations were finished the muck of Stage 3 was removed, and excavation for the remainder of the power house was continued. The work progressed from the top down in steps or benches. Line drilling with holes at 12-inch centres was performed along the power house walls where neat surfaces were required. As shown in Figure 7, the excavation progressed so as to allow much of the rock to be removed via the main access tunnel. Ramps were established with maximum grades of 15 per cent and all hauling was done with diesel powered 15-ton trucks. The trucks were loaded by 1-1/2-cubic yard diesel shovels. Excavation was halted in the power house at the invert level of the penstocks and driving of the penstocks started. Work was resumed in the power house when the horizontal portions of penstocks Nos. 1, 2, and 3 were completed, and thereafter excavation continued simultaneously in both locations. Draft tube No. 8 was completed soon after work started on the penstocks. The ramp to the power house access tunnel was removed and thereafter all rock was hauled to the surface via draft tube No. 8 and the tailrace. Successive stages of power house construction are shown in Figures 8, 9 and 10.

Excavation of the tailrace chamber was carried out at the same time as excavation of the power house. The roof was rock-bolted where necessary and covered with a 4-inch layer of reinforced gunite as work progressed. The first stage of the tailrace chamber excavation (Figure 7) was continued from the access tunnel along the top of the tailrace tunnel until the surface was reached. This provided an additional means of access to the tailrace chamber through which all of the remaining tailrace rock was removed. Draft tube No. 8 was driven through to the power house. Draft tubes Nos. 1 to 7 were driven from the tailrace in the sequence shown on the figure.

Most of the concrete in the power house was placed by pumpcrete equipment from an installation in the tailrace chamber. Wood forms were used throughout.

The total installed capacity of the development will ultimately be 1,200,000 horsepower. Four 150,000-horsepower vertical hydraulic turbines directly connected to 138,000-kva generators have been installed and are in operation, while installation of the four additional units for the final stage of development is now nearing completion. A typical section through the power house showing the units is presented in Figure 6.

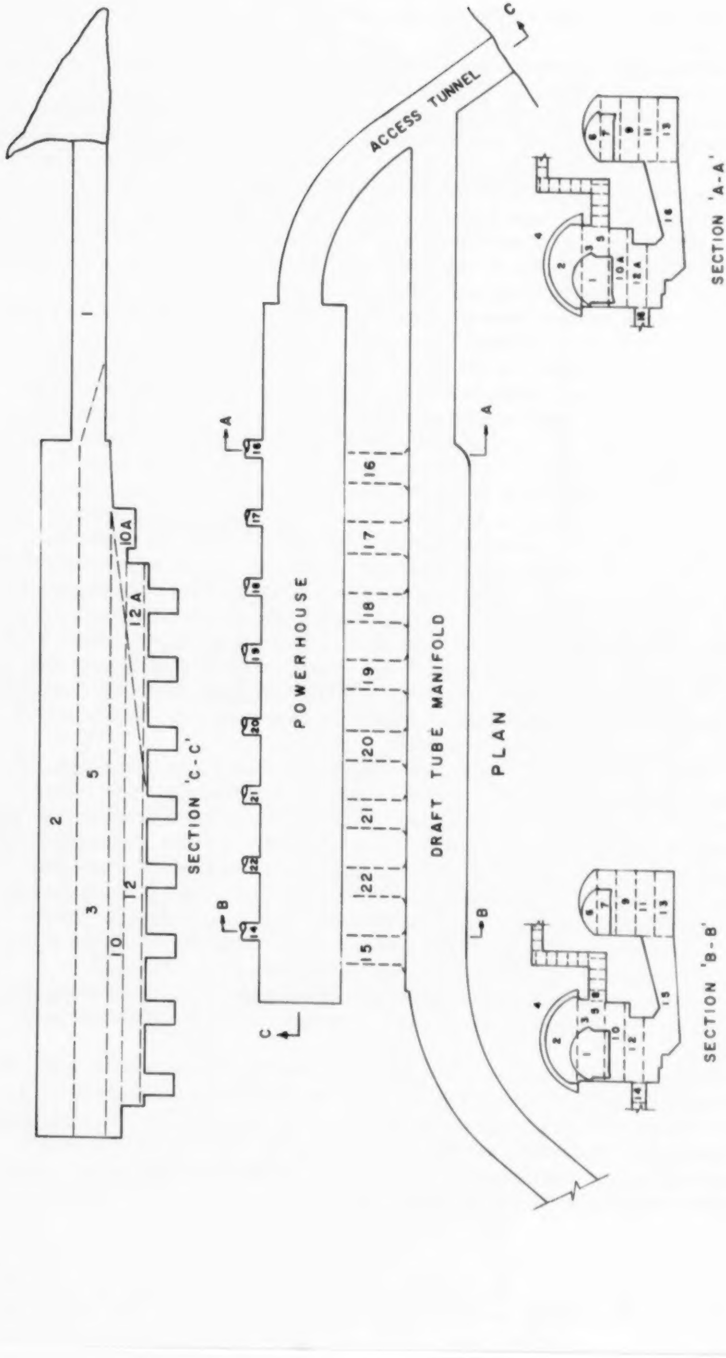

FIG. 7 BERSIMIS No.1 POWERHOUSE, EXCAVATION SEQUENCE

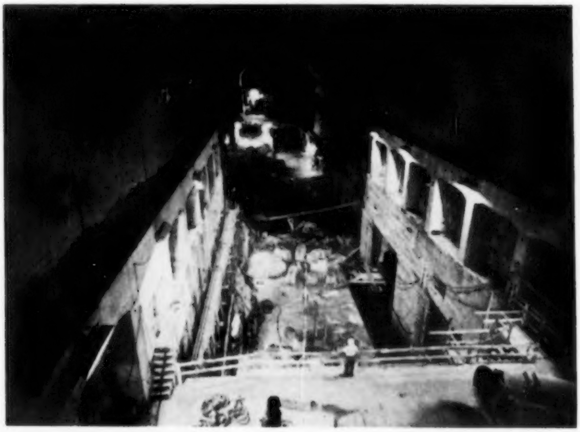

FIG. 8 BERSIMIS No. 1 EXCAVATION AND FIRST STAGE CONCRETE

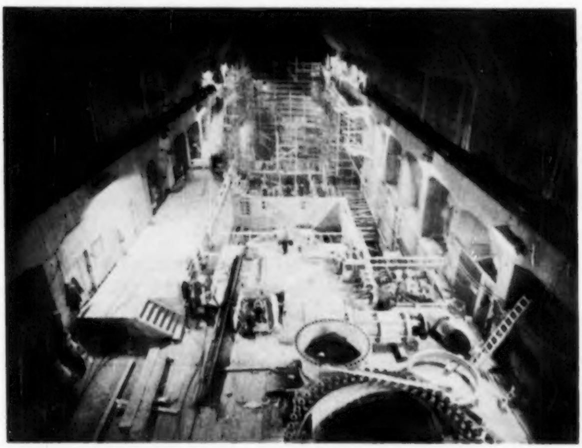

FIG. 9 BERSIMIS No. 1 SECOND STAGE CONCRETE

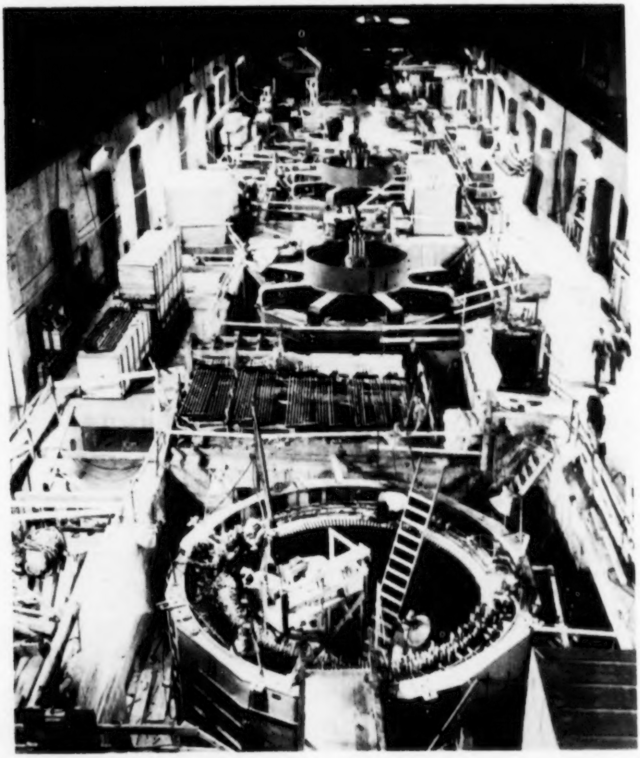

FIG. 10 BERSIMIS No. 1 GENERATOR ERECTION

Water is introduced into the turbines through hydraulically operated penstock valves of the straight flow type. A 400-ton double hoist power house crane is provided for erection and maintenance of the turbines and generators.

The power from the generators is delivered through open bus work running through tunnels, one tunnel for each unit, and then by isolated phase bus to transformers located in an outside transformation and switching station. The turbine generator units are controlled from a building in the transformation and switching station which is connected to the power house by a combined access and control cable tunnel.

Ventilation of the power house is accomplished by circulating air from the tailrace through a ventilation tunnel by means of blowers in the power house, and out through the bus tunnels. Provision is also made for recirculation of air.

From oil-filled, fan-cooled 13.8/301.4-kv single phase transformers, power is dispatched through the switching station into the transmission lines leading to Quebec City.

(c) - Chute-des-Passes

The Saguenay Power System owned by the Aluminum Company of Canada Limited, at present consists of five major hydro-electric plants on the Saguenay and Peribonka Rivers. In 1941 the Aluminum Company of Canada Limited completed construction of the Passe Dangereuse dam on the Upper Peribonka River, providing additional regulated storage for the Isle Maligne and Shipshaw power developments downstream on the Saguenay River. Additional controlled storage for these developments is provided at Lake Manouan by a timber control dam.

To augment the Saguenay Power System and to provide for additional potroom capacity at the Arvida smelter, the Aluminum Company of Canada Limited have started construction on the Chute-des-Passes project, which will develop power from the head created by the Passe Dangereuse dam. The gross head will vary between approximately 540 feet and 640 feet and the net head will vary between 470 feet and 635 feet. Construction commenced late in August, 1956. Initial power will be produced at Chute-des-Passes in August, 1959, with the remaining units coming into service at two-month intervals.

The development is located on the Peribonka River immediately downstream from the existing Passe Dangereuse dam, approximately 90 air miles north of Isle Maligne. It will be an underground installation incorporating a concrete-lined supply tunnel, approximately 31,000 feet long, leading from an intake structure at the Passe Dangereuse reservoir to an underground power house. The power house will be located approximately 400 feet below the surface of sound bedrock and will house five units with a capacity of 200,000 horsepower each. Water discharged from the power house will enter an unlined tailrace tunnel approximately 9,000 feet long. Incorporated with the supply tunnel will be a vertical surge shaft leading to a surge tank located approximately 2,800 feet upstream from the power house. In addition, to accommodate surges in the tailrace tunnel, a surge chamber will be located immediately downstream from the power house, providing a manifold for the draft tubes. Plan and profiles of the development are shown on Figure 4.

Construction of the intake will be carried out without interfering with normal operation of the Passe Dangereuse reservoir. The intake structure

will have two openings, will be built in sound rock, and will incorporate two sets of trash racks and two head gates. An intake tunnel varying in diameter from 50 feet to 70 feet and some 600 feet long will lead from a portal in the shore of the reservoir to a small forebay immediately upstream from the intake structure.

The power tunnel will have a nominal inside finished diameter of 35 feet and will be of horseshoe shape. The concrete lining will be approximately 1-1/2-feet thick. Concrete will be dropped through a down hole at the mixing plant, located about halfway along the length of the tunnel, to a hopper in the tunnel. From this point the concrete will be transported to pumpcrete machines located near the point of placing, and will be pumped into place behind the movable steel forms.

A concrete-lined vertical surge shaft 30 feet in diameter will lead from the supply tunnel to a surge tank constructed in the rock and located approximately 2,800 feet upstream from the power house. A surge tank having a nominal diameter of approximately 130 feet is required to provide operating stability. The lip of the surge tank will be set so that with full load rejection from more than one unit, a proportion of the water will be spilled from the surge tank.

Five steel-lined penstocks will lead from a reinforced concrete-lined manifold located at the end of the power tunnel. In the vicinity of the power house, the steel liners will be designed to sustain full static head plus pressure rise without any support from the rock. Upstream from this section the liner thickness will be decreased as the distance from the power house increases, so that a greater proportion of the total internal water load is resisted by the rock and less by the steel. The penstocks will be horizontal and will vary in diameter from 15 feet to approximately 11 feet. It is contemplated that the power house will be similar in arrangement to that at Bersimis No. 1, but will be slightly larger in cross section due to the greater physical size of the turbines and generators. A typical section through the power house is shown in Figure 11. The power house will house five vertical Francis type turbines, each having a maximum capacity of 200,000 horsepower, direct-connected to five generators of 165,000-kva capacity. Inside the power house, immediately upstream from each turbine, a spherical type penstock valve will be provided. The power transformers may be located directly above the power house at the surface. The low-voltage leads from the generators will be led through vertical bus ducts from the power house to the transformers. The power house will incorporate an erection bay, and two overhead power house cranes will be provided which together are capable of handling the heaviest single piece of power house equipment.

The draft tubes will lead to a draft tube manifold which will be designed to form a surge chamber to accommodate surges in the tailrace tunnel. Access to the power house will be gained through a 2,600-foot long adit leading from the surface at a grade of 7 per cent, and this adit will incorporate a branch adit leading to the draft tube manifold.

The tailrace tunnel, which will be 48 feet in diameter and unlined, will lead from the downstream end of the draft tube manifold and will run full under all conditions of operation. The tunnel will run to a point some 9,000 feet downstream from the power house into an open channel tailrace discharging into the Peribonka River.

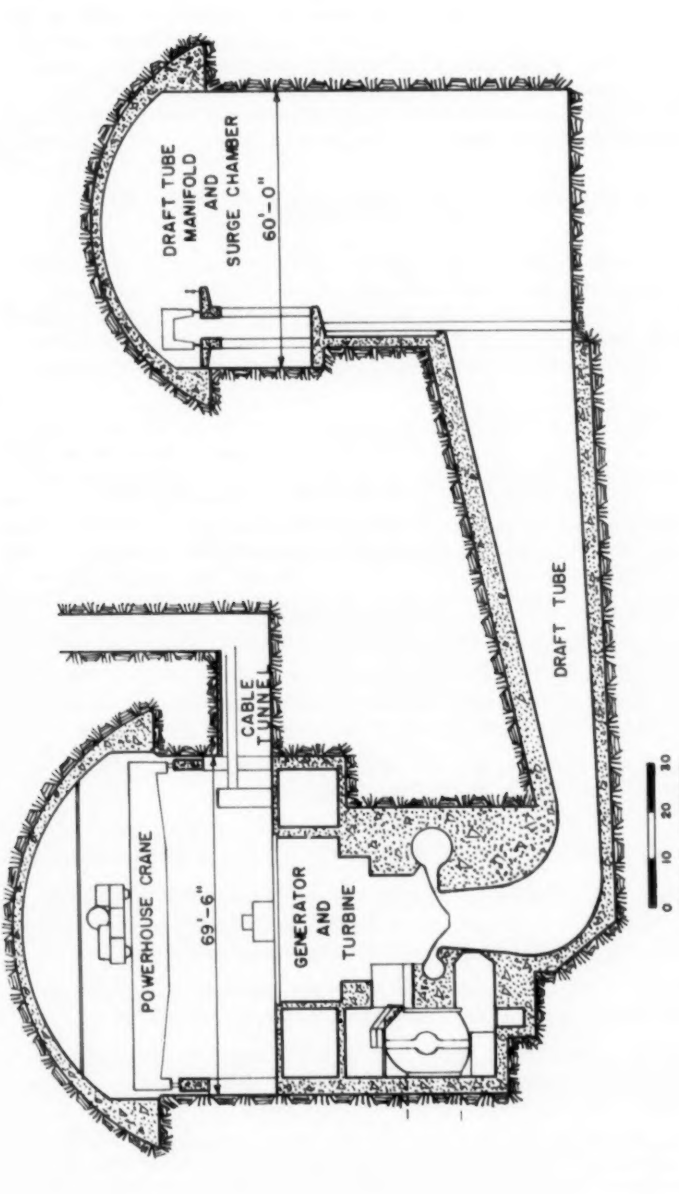

FIG. 11 CROSS SECTION OF CHUTE-DES-PASSES POWERHOUSE AND DRAFT TUBE MANIFOLD

The switchyard will be located on the surface immediately above the power house and will contain the power transformers, reception building, and all the necessary high-voltage switchgear. It is planned to build up the whole switchyard area using rock excavated from the power house and tail-race tunnel. The use of aluminum for the switchyard structures is being examined and comparative estimates are being prepared for steel and aluminum structures. Power will be transmitted from Chute-des-Passes at 345 kv nominal, 400 kv maximum.

ACKNOWLEDGMENTS

The data concerned with the Bersimis developments and the Chute-des-Passes developments were made available, respectively, by the Quebec Hydro-Electric Commission and the Aluminum Company of Canada Limited.

The authors wish to express their thanks to the engineers of the above-mentioned organizations who made possible the preparation of this paper.

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ROCKFILL DAMS: KENNEY AND CHEAKAMUS DAMS

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(Proc. Paper 1671)

FOREWORD

This paper is one of a group from the Symposium on Rockfill Dams, June 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element and is described approximately as follows: a dam in which at least 50% of the maximum section is quarried rock; and in which at least half of the rock is dumped from lifts rather than placed in layers. This includes the types with impervious face membranes, sloping earth cores, thin central cores and with thick cores as limited roughly by the above description.

The objective of the Symposium is to assemble up-to-date information on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Site and materials data, design criteria, construction procedures and some performance records are presented in this paper on two zoned rock fill dams in British Columbia. One is founded on rock and the second on a mud slide.

INTRODUCTION

Fill dams are the natural choice for first consideration by hydraulic engineers in British Columbia when required to design an impounding

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structure and of the several types, rockfills are preferred because rock is almost always available in ample quantities, whereas the fine grained sediments required for an earth or even thick core dam are quite often in short supply. The geology of the stream valleys in particular is featured by the effects of repeated, long term and intense glaciation, which now appear as extensive filling or plugging of the ancient channels by drift and till. With few exceptions the modern all-rock dam sites are associated with plugs so high and resistant as to divert the streams permanently over rock through which they have eroded canyons in post glacial times. Kenney Dam site is of this type, while Cheakamus Dam rests on unconsolidated valley fills.

The criteria for a rock fill dam appear to be amply satisfied by Kenney, but Cheakamus calls into play definitions of the term.

The editors of Engineering News Record in the issue of October 24, 1957 proposed to define a rock fill dam as one made up of more than fifty percent rock. However, Mr. A. L. Alin, by letter appearing in that publication November 28, 1957, suggested the following: "A rock fill dam is a type of dam which, for its stability, depends on loosely dumped rock." It appears that any definition must be broad and relatively loose - and should establish the nature of a rock, as well as the manner of use and the purpose served. A fill of coarse gravel would be built in the same manner and have all the qualities and characteristics of a dumped rock fill but should probably not be designated in those terms. If Mr. Alin's definition is narrowed by adding the words ". . . of quarry shapes and sizes" then it is both convenient and accurate to classify Cheakamus as a rock fill dam.

Vogues or schools so evident in the history of the engineering construction art also manifested themselves in British Columbia, except that dam construction on a larger scale began there when concrete was a preferred medium. Consequently the current success of rock fills in that area is entirely a post war development, and based on the following economic factors:

1. Utilization of more remote sites for storage and power development which involve longer supply lines and transport costs as elements of total construction cost. With rock available locally, capital cost becomes less than for an equivalent concrete dam.

2. Great improvements in quarrying equipment and techniques, and the steady growth of heavy excavation and haulage units.

3. Practical elimination of maintenance expense during operating life of the dam, which is an increasingly important element of cost, generally accentuated in mountainous locations by difficulties of access and more severe weathering action by high altitudes.

4. In many cases, rock-fill dams can be built on the natural river bed materials so often found at topographically favourable dam sites, because high velocity stream flows have carred away the fines and reworked the coarser particles into the equivalent of a first quality dumped rock fill.

In addition to the several cost advantages offered by rock fill construction in certain locations, these embankments provide desirable inherent toughness and reliability. They have the capacity of self healing under shock as from an earthquake or bombing attack, and are able to pass a large amount of leakage without danger of failure or even permanent damage. This latter

feature of dumped rock embankments suggests the concept of rock fill dams as the ultimate development of rock or gravel toe drains commonly incorporated in earth dams.

Performance of Kenney Dam

Kenney Dam, located in the central part of British Columbia on the Nechako River and containing a total of about four million cubic yards of all materials, was installed to impound about twenty million acre feet of water for the Kemano power plant of the Aluminum Company of Canada (Alcan) B. C. Development. Reservoir flood spillway and power intake are remote from the dam which has a hydraulic height of about 275 feet to top reservoir elevation 2800, and a total height of 325 feet above the low point of the foundation.

The section is of the sloping core type with finished upstream slope of 2.5:1 and average downstream slope of 1.75:1. Width of crest is 40 feet and minimum freeboard above full reservoir is 20 feet. Length of the dam at the top is approximately 1600 feet, arched in an upstream direction to a radius of 5729 feet, (1° curve).

Site

The damsite is in a post glacial channel eroded by the Nechako River through a series of basalt flows dipping about 7° in a downstream direction and forming the upper end of the Grand Canyon of the Nechako. At this point the river which carried discharges ranging between 1500 and 30,000 cusecs, flowed in a steep walled canyon about 80 feet deep on a gradient approximating 3%. Above the rim of this rock canyon the sides of the valley were burdened with varying depths of glacial till eroded to slopes of about 35°.

Design

The adopted design closely followed that developed for the Nantahala Dam of the Aluminum Company of America, except for the flattening of the downstream slope to add safety in the event of earthquake shocks. The general principles of this design are to provide a supporting member of dumped rock, a graded filter on the upstream slope of the rock section, a rolled fill sloping core, a superimposed filter of sand protected in turn by pitrun gravel and an upstream surface layer of quarry run rock to absorb wave action. At Kenney, the upstream cofferdam was incorporated in the blanket covering the rolled fill core. Abutment contacts downstream of the blanket were everywhere stripped to rock, but the boulder-gravel deposit in the original river bottom downstream of the core and filter contacts was accepted as foundation for the dumped rockfill. Plan and typical section are shown on Fig. 1.

Construction Materials

Rock fill was obtained from strata of massive basalt in the left abutment, also most of the 3" - 10" downstream filter zone was produced by crushing and screening reject from the quarry. Scattered deposits of alluvial origin and mostly on the right bank supplied all sands, some filter gravel by screening, and the pitrun fill overlying the core. These natural sands and gravels were poorly graded and in relatively small pockets which led to a regular

program of exploration and pit development in order to maintain production, and eventually the haul distance from pits was in excess of two miles. Impervious fill material was borrowed from a large deposit of boulder till on the right bank and about a mile downstream from the dam axis. As explored by a pattern of augur holes 20 feet in depth and several test pits, this till average approximately 10% clay sizes and contained about 8% of water.

Construction

The Nechako River was diverted around the left abutment through a 32 foot horseshoe tunnel about 1500 feet long, by an upstream cofferdam approximately 90 feet high. When unwatered, the river bed proved to be a maze of rock pinnacles, deep holes, and scour channels. The core contact area on the abutments was trimmed to slopes not steeper than 1.5:1, gunited to seal the surface of the vesicular lava and broken basalt, and a grout cutoff was constructed by drilling through the gunite seal coat. At some points, concrete was used to build out re-entrant steps in the core contact area. The gunite seal coat proved capable of withstanding shallow consolidation grouting pressures of about 30 p.s.i. and forced some horizontal distribution of the grout.

A foundation grout curtain located on the center line of the core contact area was completed by drilling about 75,000 linear feet of holes which took a total of 48,000 bags of cement at pressures up to 200 p.s.i. Depths of holes in the sequence were respectively 30', 75' and 150'. Downstream of the core contact, the abutment rock was trimmed to 1:1 slope as far as the axis of the dam, thence warped back to natural slopes within the limits of rock fill contact.

A base for the impervious core was constructed of concrete, filling in the extremely irregular rock surface of the river bed and finished to a level surface about 80' wide and 100' long at El. 2520 on which the first of the clay fill was compacted.

To produce rock fill material, a face about 1500 feet long was developed in benches by coyote hole quarrying, requiring slightly less than an average of one pound of blasting powder per cubic yard broken. Despite very favourable cores from exploratory drilling of the basalt deposits, an unexpected amount of residual clay and fines were produced by each blast, which required selective loading of the product in the quarry and constant attention to allocation of the loads. When the fines content of a load exceeded 15% by volume, it was dumped down a high face, with resultant segregation of part of the rock content which was then reclaimed along the toe of this slope. Other loads considered unsuitable for the main fill were put through a crushing plant and screened to produce the coarser filter sizes. All except the upper portion of the rockfill was built out from the left abutment in lifts approximately 40 feet high, with each load sluiced down the slope from the top of the lift by skid mounted monitors delivering about two parts of water to one part of rock. Most of the sluicing water was reclaimed at large sumps just inside the downstream cofferdam and pumped back through the monitors.

Filters

The three filter zones downstream of the core were placed by truck in 10" to 12" lifts, watered and compacted by caterpillar tread tractors except that the sand zone for 100' from each abutment was spread in thinner layers and

given extra compaction. The zones were brought up in steps, each about five feet higher than the adjacent upstream zone. Two patches of a fourth filter of 1/4" - 3/4" sizes were placed above the principal break in the rock abutments as trimmed, to increase the safety factor during settlement of the completed embankment. A large portion of the screened and stockpiled sand for the fine filter zone contained so much silt that when moistened in layers on the dam, it became somewhat plastic and was insufficiently free draining; consequently that fraction which contained more than about 2% of silt had to be washed before hauling to the embankment.

Impervious

The boulder clay used for the sloping core or impervious diaphragm was first broken up with rippers to about 30" depth, then moistened in the pit by a dozer equipped with raker teeth and hauling a tank wagon, feeding water to the point of each rake tooth. This irrigation plus dozer harrowing distributed the water through the loosened till, which was then allowed to temper for several days before loading into pushed scrapers for transport to the dam, where it was placed in 10 inch lifts, and compacted by 16 passes of sheepsfoot rollers to 95% of Standard Proctor density. Despite the processing in the pit, some hand picking of oversize (6") rock was required after spreading on the embankment. Except at contact with the abutments, the core was placed on the dry side of optimum, that is, about 12% moisture by weight, which gave a compacted dry weight averaging 110 lbs, per cu. ft. At base and abutment contacts water content was somewhat increased, rocks larger than 2" were removed, and compaction was accomplished by pneumatic tampers.

The pitrun sand and gravel overlying the upper sand filter was placed with inter-bedded pervious zones to assure freer drainage under drawdown conditions. It should be noted, however, that drawdown from full reservoir level will probably never exceed 20 feet.

The dam proper was built in the relatively few months between May and November 1952, at peak rates of 50,000 cu. yds. place measure of all classes per day of two shifts. Reference is made to H. Jomini's article in the Engineering Journal, November 1954 for additional details of construction.

Two sets of strain measurement points were embedded about 4.0' below the surface of the fill and at spacings of 100' along the base line. The series of 13 monuments near the top of the core initially were on a vertical camber or curve of 3.5' middle ordinate. The second set of 11 monuments along the upper berm of the rock fill were initially in a horizontal plane at el.2782.75 or about 36' below the crest monuments.

Performance

The time rate hydrostatic loading of Kenney Dam is shown in Fig. 2 starting October 8, 1952 and reaching full reservoir elevation 2800 early in 1957. Although the site presented the possibility of leakage through the abutments along contacts between successive lava flows beyond the limits of the foundation grout curtain, no seepage has been detected coming from the abutments or through the embankment proper. The vertical settlement and horizontal movement both downstream and longitudinally at each of the two sets of monuments have been measured six times since December 1952, most recently in July 1957. The direction and magnitude of these deflections or

movements are illustrated by Fig. 2, representing data obtained through the courtesy of Power Operations Division of Alcan, B. C.

The graphs were plotted using only the vertical and downstream displacements of crest monuments #6, 7 and 8, recording for the central 300' of dam length and averaged to smooth out any effects of probable errors in surveying and possible other localized movements. After the first eleven months, the approach of the graphs to the horizontal is quite noticeable but an increased divergence was reported for the year 1956 to 1957. Theoretically, this angle should decrease to near zero in another decade. The largest vertical settlements, which have occurred near the maximum section of the dam, are approximately 1.5 feet or 30% of the vertical camber provided at that point.

While the vertical and downstream or offset movements conform to expectations, the observed lateral or cross-valley displacements of the plates are inconsistent with theory, even after making all reasonable allowance for possible errors of measurement. The center point of a fill in a symmetrical valley should hold its original cross-valley position with reference to fixed points at the ends of the dam, during vertical and downstream displacements, likewise intermediate points if compressive or tensile strains are uniformly distributed. At Kenney Dam both on the crest and along the berm, the total lateral movement to date approximates 6" at the center reducing to about 2" near the left abutment, and consistently towards that end of the dam.

Consequently, a particle has moved vertically downwards, horizontally downstream, and laterally towards the left abutment, hence the resultant path could be represented by the diagonal of a parallelopiped.

No maintenance work of any kind has been required to date and none is likely to be necessary in the foreseeable future.

Cheakamus Dam

The Cheakamus Dam completed in 1957 is located on the Cheakamus River, about 25 miles north of the head of Howe Sound in British Columbia, as part of the 190,000 HP, 1125' head Cheakamus Development of B. C. Electric Company Ltd. The structure has a maximum height of 88 feet above foundation and will impound about 40,000 acre feet for release through a tunnel intake nearly one mile distant. The right abutment of the embankment is a combination gravity concrete diversion and spillway structure for discharging about 45,000 cusecs maximum.

Site

The normally complex geology of the area is in this instance further complicated by a large mud or landslide which originated on a flank of Mount Garibaldi about 100 years ago and stopped as a flattened cone or fan damming the Cheakamus River to form Stillwater Lake. The conditions imposed by this event became the dominant feature in the engineering and construction of Cheakamus Dam.

The slide followed the valley of Rubble Creek which was then probably in a rock canyon, and overfilled the alluvial and bedrock channel of the post glacial Cheakamus River. The foundation of the dam, therefore, in succession upwards is an extremely broken and irregular granite base, followed by glacial and alluvial sand and gravel overlain by varying depths of the landslide material which is hereafter called Rubble Creek Wash. In the eastern

portion of the valley basalt is in contact with the granite base or with mantle on the granite.

Drill holes showed wash material in excess of 200' thickness. The slide broke off large trees of the living forest, enveloped the stumps and trunks and swept the tangle of loose forest growth towards the terminal contacts with the western slopes of the valley. Since the date of the slide the river had cut down into the wash about 35' and lowered the lake level.

Matters of concern in the investigation of the site included the following possibilities:

1) Under seepage through talus along the rock walls of the earlier valley.

2) Under seepage through the alluvium beneath the wash.

3) Under seepage through zones between the granite and superposed basalt.

4) Under seepage through fractures in the granite.

5) Under seepage through porous buried forest between the wash and alluvium.

6) Deficiency of strength and watertightness of the Rubble Creek Wash itself.

In all, nearly 100 exploratory holes were drilled to trace out the patterns of the foundation and test pits were dug in the wash, one to a depth of 40 feet.

Design

One of the earlier axes investigated which incorporated an outcrop of basalt surrounded by wash was eliminated because of probable large talus accumulations imperfectly sealed by landslide material. The site adopted is just upstream of the section where the river had cut to the rock forming the indicated right wall of the pre-slide channel. This placed the entire embankment on Rubble Creek Wash except at the right abutment contact with granite and the channel portion of the rock fill. In succession from the right end to the left end of the dam axis the drill holes showed thicknesses of Rubble Creek Wash to be 0', 62', 15', 35', 54', 122', and 126'.

This location utilized the upstream area of the slide fan as a blanket, eliminated a saddle dam, simplified stream diversion and reduced the overall length of structures. As site investigation progressed and the ground was opened up for construction, questions of water loss from the reservoir were resolved favorably. Several trenches were cut along the contact of the wash with the right or west wall rock of the valley, and filled with water. The losses were so small as to obviate special sealing treatment. Along the left side of the reservoir, the wash extended over possible talus up to full reservoir level, and sufficiently lengthened the path for seepage from farther upstream.

Adequacy of cutoff on this side was assured also by the great depth of consolidated wash in the pre-slide river canyon and extending well up the Rubble Creek Valley. Similarly, any seepage paths through the alluvium were considered to be sufficiently lengthened by this heavy blanket, which was found to have settled into tight contact with standing tree trunks. Grouting and water tests showed that jointing in the granite would introduce no hazard.

Since the slide material would be weak and unstable when again saturated, flat slopes were selected, and the upstream toe in the channel was buttressed by additional fill and the cofferdam.

The design of the dam includes a downstream supporting element of dumped rock with the center of this rockfill bearing on a ledge exposed in the natural streambed. Upstream of the rockfill and three filter zones, the impervious member is made up of Rubble Creek Wash founded on the same material as present in the streambed and left abutment, and on the old granite of the right abutment. This section was finished to a 4:1 slope down to el. 1190 then carried level about 200 feet to contact with the upstream cofferdam. Depth of the slide material at the upstream end of the impervious section is in excess of 200 feet. The dam was overbuilt to a crest camber of 2' maximum as allowance for settlement.

The rock section was given an average downstream slope of 1.6:1 and is underlain by sand and gravel filters and a system of tile drains. The filters provide against the migration of fines from foundation wash or along contacts between wash and ledge rock. Seepage collected in the system of under drains is trapped behind a small impervious dike at the toe of the maximum rock section and discharged through a gauge house equipped with measuring weir. Because of the possibility of leaks through the shattered granite below the mudslide material and earlier overburden, permanent drainage wells were installed at two points for observation and measurement. A plan view and maximum section are shown as Figure 3.

Construction Materials

Rock fill was obtained from stripping the river bed, excavations for diversion channel and concrete structures, oversize rock from grizzly screening of Rubble Creek Wash and from talus accumulations along the valley wall. Sand and gravel for filter layers was produced by screening river deposits, while the "quarry run" or 4" maximum size rock for blanket was selected from spoil piles at the site or at the east portal of the power tunnel. The Rubble Creek Wash material for the impervious zones was shovel excavated and trucked from pits near the south end of the dam, which yielded a typical slide mixture of sand, silts, a very small amount of clay and many small to large sub-angular rocks representative of the Garibaldi Mountain extrusives. The natural water content of this mixture was just above optimum and the permeability coefficient is 10^{-3} to 10^{-4} cm per second.

Construction

After excavation of a diversion channel through the rock of the right abutment and completion of an upstream cofferdam of rock and Rubble Creek Wash, the foundation area of the dam was prepared by stripping the surface to fresh unweathered wash where this material would become the foundation, removal of loosened rocks and exposed concentrations of rock and compaction by loaded scrapers. Rock along the right abutment and in the river bed was stripped clean and areas to be in contact with the impervious zone were gunited to fillet the re-entrant corners and otherwise smooth the surface. In addition a curtain of pressure grouting was extended from the gravity concrete sections of the dam upstream parallel to the diversion channel cut.

Springs in the foundation area of the impervious zone were piped to sumps - extended upwards with the embankment and finally backfilled with selected

soil through which the pipes were plugged by pressure grouting. Similar springs in the rock fill area were confined by dikes and piped permanently by a tile line in graded filter to a gage box at the downstream toe.

In the area of the dam, contact zones between bedrock and wash were cleaned out by trenching and sealed with a layer of filter capped by selected Rubble Creek Wash.

After placing drain pipes, spreading and compacting a 3' layer of fine filter followed by 2' of corase filter and a thin lift of compacted small rock, the dumped rockfill was brought up in approximately 20 foot lifts, timed to keep ahead of quarry run, coarse, and fine filters in that order upstream towards the impervious zone. The filter between the dumped rock and wash of the left river bank was made up of 15' of fine filter blanketed by 3' of quarry run. Since the sloping filter zones were too thin to be placed by dumping from trucks at grade, the contractor generally utilized a high lift loader or small dragline to build them by rehandling stocks of the material delivered by truck from the top of the rock fill. Fine filter sizes ranged between #40 and 1" and coarse filter between 1/4" and 1" or 2". No sluicing water was used on the dumped rock.

First lift in the impervious zone consisted of three 8-inch layers of Rubble Creek Wash which had been grizzled to remove stones above 6 inches in size. For compaction at contact with rock or concrete the pit material was similarly screened to exclude stones more than 4 inches in size. Above these foundation lifts the Rubble Creek Wash was taken directly from the pits, excluding rocks more than 18 inches in dimension, and compacted in 24 inch layers by hauling equipment. A loaded scraper was used to compact the foundation layers.

A large portion of the rip rap for the upstream slope was supplied by raking out oversize rock from the Rubble Creek Wash as it was spread on the embankment. Due to the rock content of the material, an equivalent armoring would have been formed by wave sorting of the mixture during several operating seasons.

The reservoir created by Cheakamus Dam reached el. 1240 in October 1957, about five weeks after the diversion channel was plugged. Seepage measured at the gauge house is practically constant at 0.10 cusecs, except for increases due to rain seeping through the rock fill to filter zones, and the structure to date appears much less pervious than expected when the project was undertaken three years ago.

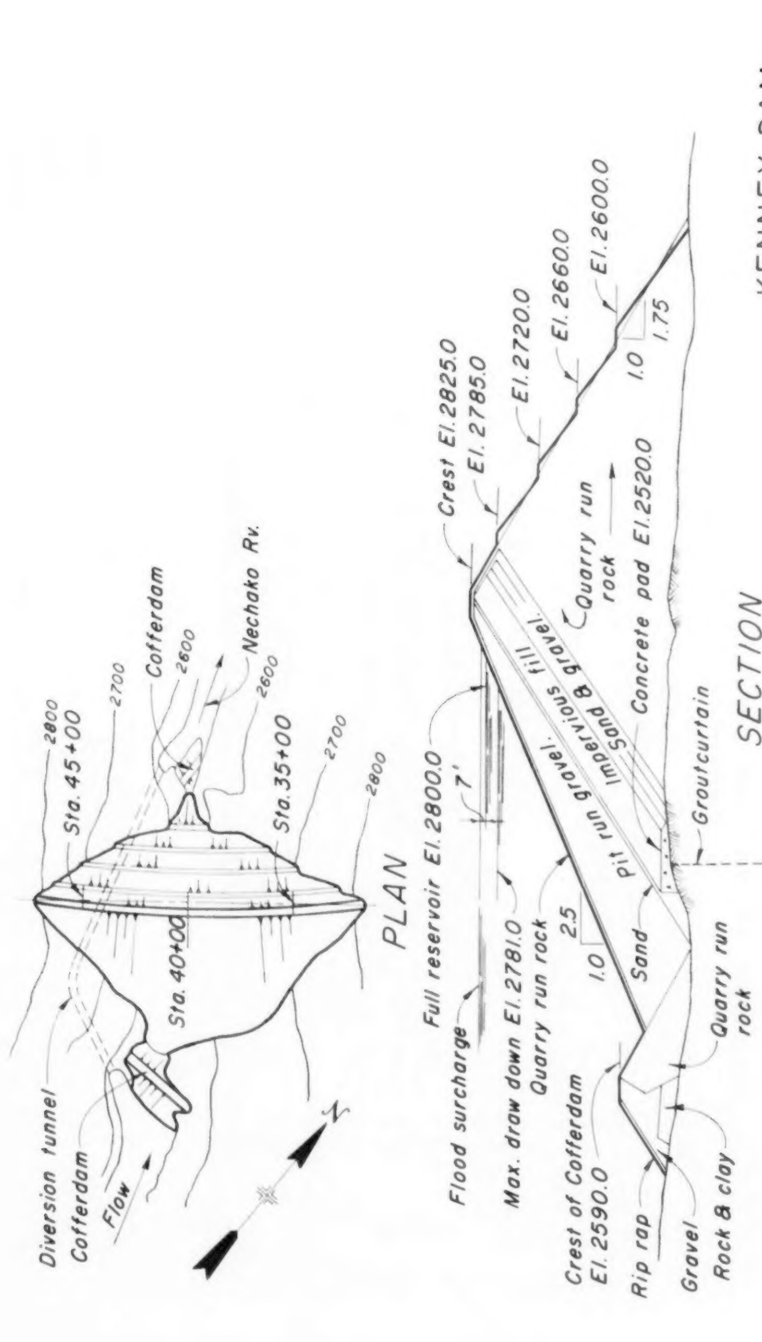
SUMMARY

The requirements for storage dams at two remote and relatively inaccessible sites in British Columbia have been met by zoned embankments of local materials. Although the Kenney site was suitable for a concrete gravity dam, the cost would have exceeded that for rockfill by about 50%. Masonry construction was not even considered for Cheakamus Dam where the special and rather complex foundation conditions could be satisfied only by a relatively flexible, self-healing blanket design.

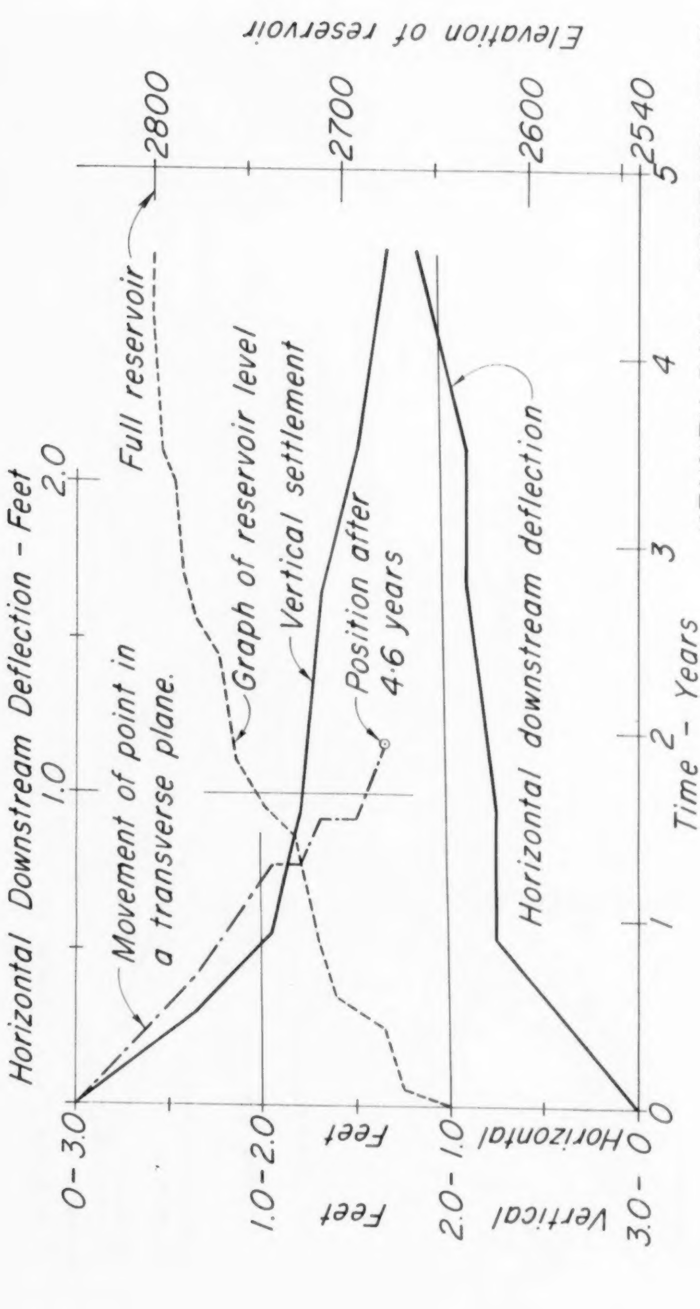
ACKNOWLEDGMENTS

Detail design of Kenney Dam was by British Columbia International Company Limited. Consultants were Doctors Karl Terzaghi and J. P. Growdon and Mr. I. C. Steel, and construction by Mannix Limited, with soils engineering by Dr. R. C. Hardy and Ripley & Associates.

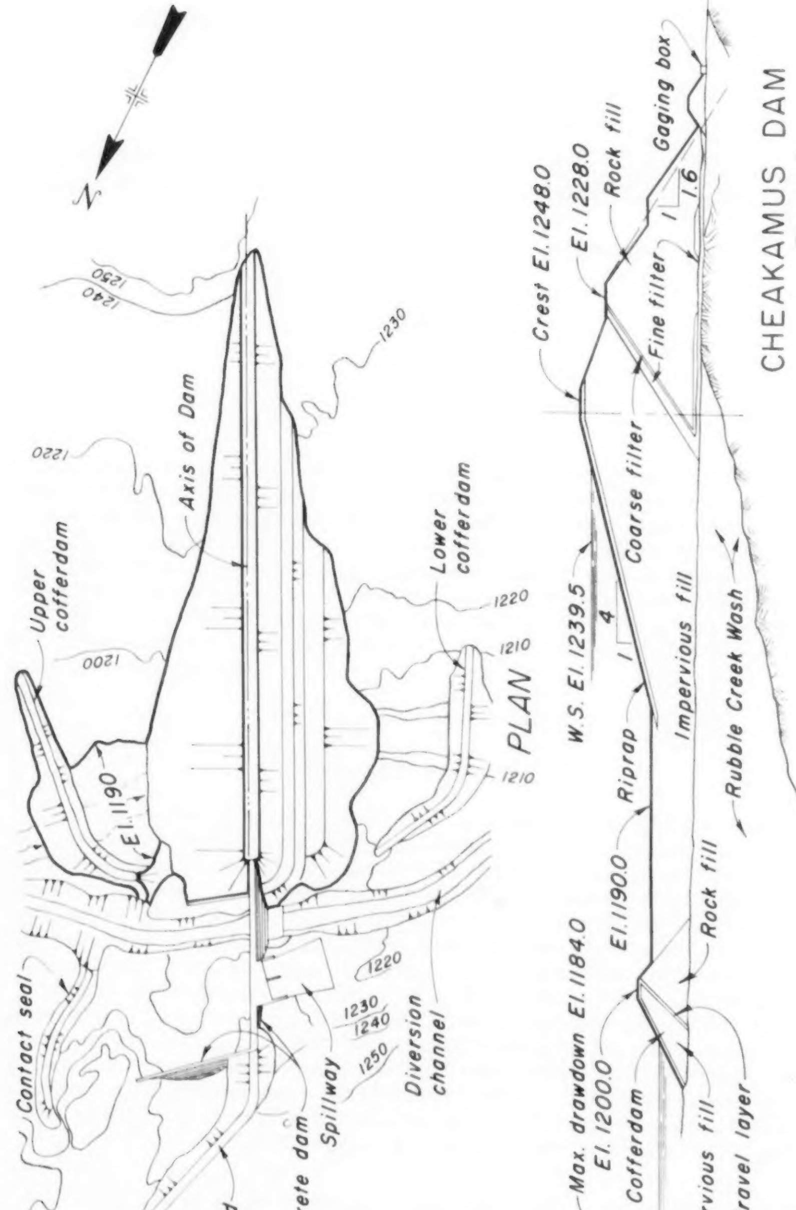
Design and construction supervision of Cheakamus Dam was by B. C. Engineering Company Limited under Dr. T. Ingledow, President and Dr. Karl Terzaghi, Consultant. Emil Anderson Construction Company built the structure and soils testing was done by Ripley & Associates.



KENNEY DAM
Figure 1.



KENNEY DAM - PERFORMANCE
Figure 2.



CHEAKAMUS DAM
Figure 3.

Paper 1672 June, 1958 PO 3

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Proceedings of the American Society of Civil Engineers

BOX CANYON HYDROELECTRIC PROJECT

Arthur P. Geuss,[1] M. ASCE
(Proc. Paper 1672)

SYNOPSIS

The unique design and construction of the Main Spillway Dam of the Box Canyon Hydroelectric Project is described in this paper, together with other project features, which include the powerhouse, diversion tunnel, forebay channel, and auxiliary spillway. The location and development of the site involved factors such as foundation, seasonal limitations on headwater elevation, and construction of the main spillway in a single low-flow season. The project is a "run-of-the-river" development, requiring the design of a dam and spillway which would not cause any increase in the natural flood water levels upstream from the project.

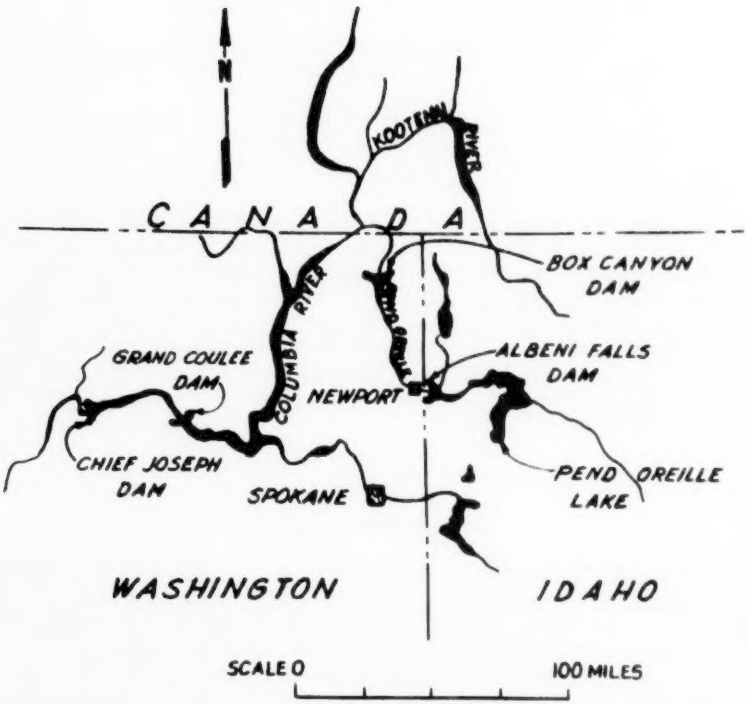
General Description

The Box Canyon Hydroelectric Project is owned by Public Utility District No. 1 of Pend Oreille County, Newport, Washington, and is located on the Pend Oreille River in the northeast corner of the State of Washington. The site is fifteen miles from the Canadian border and 90 miles north of the City of Spokane, as shown on Fig. 1. Construction was started in September, 1952, and completed in the fall of 1955. The first generating unit began operation in June, 1955, with the remaining three units following shortly thereafter.

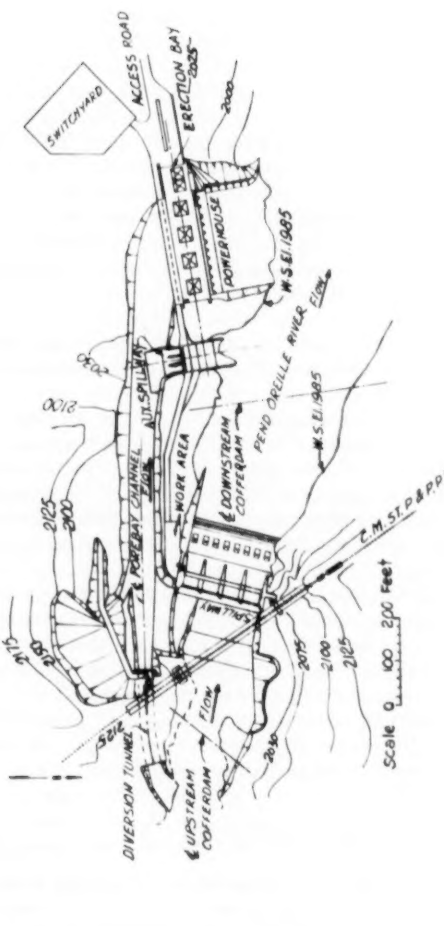
The principal structures, shown on Fig. 2, are the main spillway-dam and powerhouse, which are connected by the forebay channel. The project has a maximum gross head of 46 feet, with a minimum operating head of about fifteen feet. Maximum turbine discharge is approximately 27,000 cubic feet per second. The powerhouse contains four vertical, adjustable-blade propeller (Kaplan) turbines, rated at 25,500 hp each at full gate, with umbrella-type generators rated at 16,667 kva, 0.9 power factor, three-phase, 60 cycle (100 rpm).

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1672 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 3, June, 1958.

1. Vice-Pres. and Chf. Engr., Harza Eng. Co., Chicago, Ill.



LOCATION MAP
BOX CANYON PROJECT
FIGURE 1



GENERAL PLAN
FIGURE 2

Site

The project is located at the downstream end of a narrow gorge or canyon with nearly vertical rock faces. Foundation rock consisted of marbleized limestone and dolomitic limestone. With an average flow of 25,900 cfs, the channel of the river is about 170 feet wide and 70 feet deep. At the spillway site, the canyon is probably a very ancient filled cavern, the roof of which has been removed by the modern river. This 150 foot deep cavern extends below the present riverbed level and is filled with fine sediments below the river deposits.

In addition to this spillway foundation problem, the site had limited space available for structures and working room during construction. At the site of the main spillway dam, the Chicago, Milwaukee, St. Paul & Pacific Railroad crosses the canyon on a bridge constructed about 1910. It was, therefore, necessary that the spillway gantry crane clear the railroad bridge.

The general location of the damsite was further limited by extensive mineral deposits located between this site and the Canadian border. To determine the most economical layout, various damsites and powerhouse locations were studied in the 3000-foot stretch of the canyon that extends upstream from the selected site.

The diversion tunnel, forebay channel, auxiliary spillway, and powerhouse were excavated from the left (south) bank, with their location being determined principally by the natural topography and the geology.

The diversion tunnel location was dictated by the location of the south pier and abutment of the railroad bridge. In order to allow room for the downstream main river cofferdam and for diversion the powerhouse was located as far downstream as foundation rock would permit. This arrangement also permitted construction of the powerhouse to proceed behind separate cofferdams, unhampered by seasonal flow variations and construction of the other features.

The auxiliary spillway was located in an area of low rock in order to minimize excavation, whereas the forebay channel had to be excavated in rock for most of its entire length and depth. Along the ridge of rock remaining between the forebay channel and main river channel, a low concrete gravity section was constructed in order to retain the headrace.

The main spillway weir and piers are supported on a concrete arch, which spans the river canyon. Upstream and downstream aprons, with steel sheet piling cutoffs, comprise the remainder of the structure.

River Hydraulics

In addition to the structures being designed and arranged to fit the site, the design of the spillway and operation of the project could not cause previous upstream river levels to be exceeded at certain seasonal flow conditions.

Between May and July of each year, river flows are high, with a peak flow of 171,300 cfs being recorded in 1948. During the balance of the year, the flow will normally range from a minimum of 5000 cfs up to 35,000 cfs, as regulated by upstream storage. The reservoir level at the dam will be carried between elevations 2010 and 2030, depending upon the flow and permissible backwater levels upstream in the reservoir. During certain periods, agricultural land cannot be flooded, while in other periods, backwater effects

are limited principally by encroachment on tailwater at the Albeni Falls powerhouse, which is located near Newport, Washington, at the outlet of Lake Pend Oreille, some 55 miles upstream. On Fig. 3, a few of the backwater curves are shown and on Fig. 4 the gross head available for various river flows and backwater limitations is shown. In general, as the river flow increases, a lower reservoir elevation is required at the Box Canyon powerhouse to maintain the same backwater effects upstream. Flows in excess of powerhouse discharge will be released through the spillway, or auxiliary spillway.

When the river flow rises to 80,000 or 90,000 cfs, gross head on the powerhouse will be reduced to about fifteen feet and turbine operation ceases. At this time all spillway gates are removed to permit unrestricted flow through this structure. A specific requirement in the design of the main spillway-dam was that it have high hydraulic efficiency with a minimum of head loss under these conditions. The spillway was designed so that it caused no increase in backwater, over that recorded before the spillway was built, during periods when all spillway gates are removed.

Power generation at the project will fit into the generation pattern of the Federal Columbia River System. Fig. 5 shows the prime power available in the Columbia River System for the water years 1936-37, which is the critical year of record. While the installed capacity of the Box Canyon Project is small when compared to the Federal System, the Box Canyon generation is available during the critical fall and winter months, when river flows in the area are usually low. During the occasional no-generation period of the June flood peak, surplus secondary energy is available from the Federal System to firm up generation at Box Canyon.

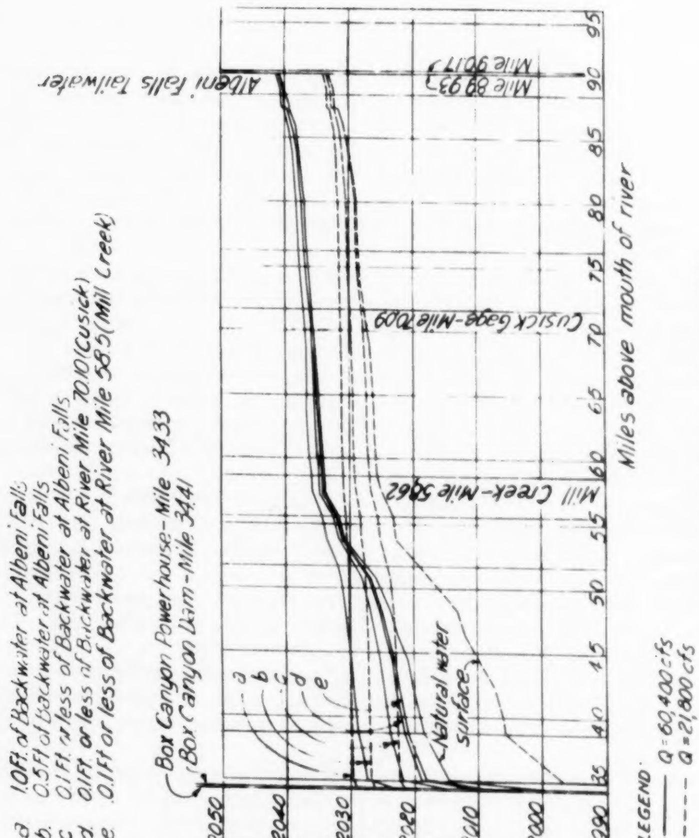
The diversion tunnel was designed for a capacity of 35,000 cfs, with upstream headwater at El. 2028. A study of existing records showed that with some upstream regulation of high winter flows, this capacity would assure almost complete freedom from overtopping the main upstream cofferdam. Limitations on upstream backwater would not permit a higher level at the cofferdam. The tunnel is approximately 225 feet long, with a reinforced concrete lined horseshoe section having a diameter of 35'0".

Minimum entrance loss at the upstream portal was obtained by use of a bellmouthed transition in the upper half of the portal and in the lower half, by excavating the approach channel to form a smooth, gradual transition to fit the tunnel section.

The invert of the tunnel was set at El. 1967 and connected with the bottom of the forebay channel at El. 1975 by means of a transition adjacent to and at the downstream end of the tunnel. This design performed very satisfactorily without an hydraulic jump and with an appreciable regain of head.

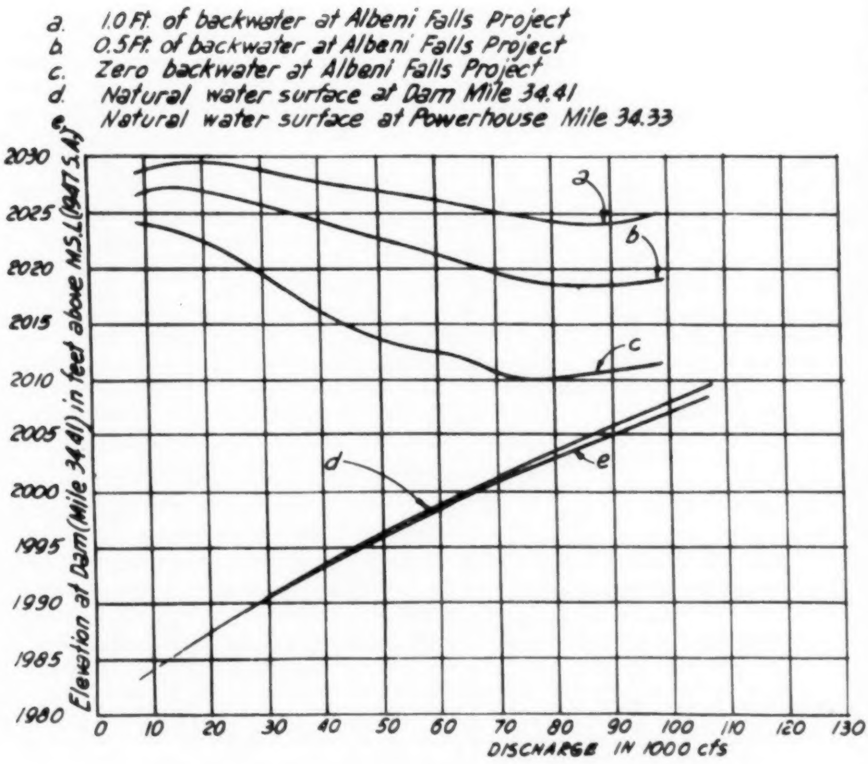
Construction

In the initial construction period, the work consisted primarily of excavation of the forebay channel and auxiliary spillway (See Fig. 2); however, tunnel excavation was started prior to completion of this work. At the upstream end of the tunnel, a narrow rock plug was left in place as a natural cofferdam, for later removal. A temporary concrete bulkhead arched in plan, was constructed on top of this plug to provide freeboard during high river stages. Initial concrete work consisted of the forebay channel wall, auxiliary



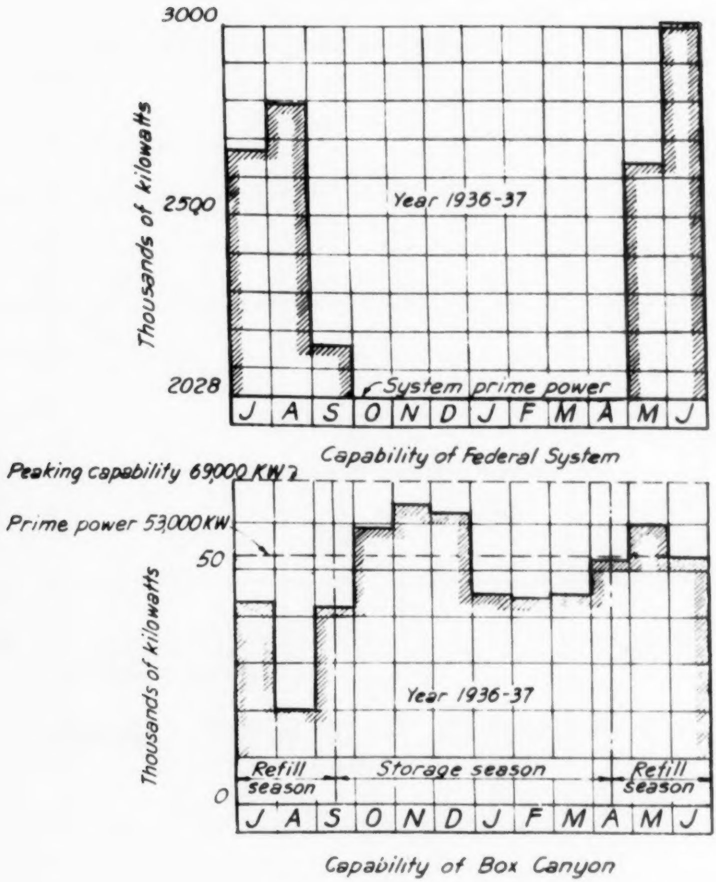
BACKWATER CURVES

FIGURE 3



HEADWATER & TAILWATER CURVES

FIGURE 4



POWER GENERATION
FIGURE 5

spillway, and diversion tunnel lining. The auxiliary spillway had been planned for construction after diversion; however, the Contractor selected it for early construction with the intention of using it and the top of the right forebay wall for an access route to the main spillway area.

Powerhouse excavation was started in this period and completed to the extent that concrete pouring began in the summer of 1953. Cofferdamming of the powerhouse area at the upstream intake side was accomplished by installing a fill cofferdam in the forebay channel between the auxiliary spillway and powerhouse intake. On the river side a portion of the rock in the tailrace area was left in place for later removal. Some fill material was also used to complete this cofferdam.

The auxiliary spillway (see Fig. 6) was primarily intended to augment the capacity of the main spillway during the larger floods and was located so that its excavation would assist in diversion. It also serves for flushing trash or floating ice from the forebay channel and for minor regulation of flows. In addition, it can serve for a portion of the intake structure, if the owner should decide to install an additional generating unit in the future and if it develops that its added flood passing capacity is insignificant. Only a small quantity of water can be discharged due to the small head available through this structure. Construction of the auxiliary spillway also required a cofferdam on the river side so that excavation could be carried down to approximately El. 1958.

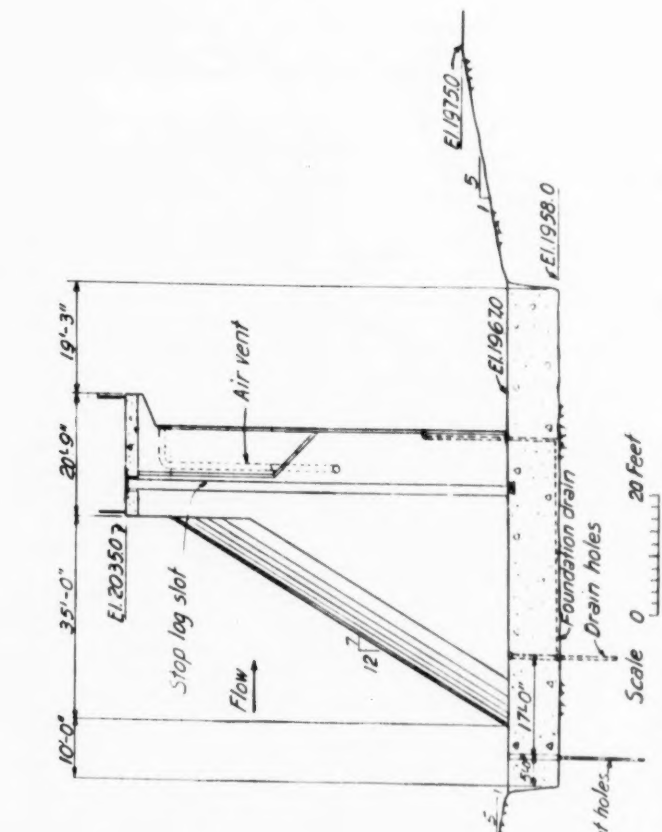
Steel stop logs were installed in all three bays between the piers of the structure, except that in the left bay, a vertical steel gate, 32 feet high by fourteen feet wide was installed at the top. This gate, operated by an electric hoist, is constructed in four sections, so that either the entire gate can be raised, permitting discharge underneath the gate, or an upper portion can be raised, permitting discharge over the top of the unraised portion. Disconnecting or connecting the section is a manual operation that can be performed from the downstream face of the gate. Fluctuations in reservoir level are seasonal, so that this operation will be performed infrequently.

The clear spacing between the piers is the same as in the powerhouse intake and the base slab is at the same elevation as the intake base slab. The present stop log slots can be used for trash racks and the remainder of the water passages for the turbine can be constructed downstream from the existing structure. The future unit centerline would coincide with the centerline of the present units and the powerhouse gantry crane would also serve this unit by extending the crane runway.

Although the actual concreting work was relatively simple, the main spillway presented by far the most difficult overall construction problem. Construction of the cofferdams, unwatering, construction of the spillway at least to above flood levels, and removal of the cofferdams had to be performed in a single low-flow season. Although the Contractor was responsible for upstream flood damages, the possible extent of such damages was so great that removal of the cofferdams at the end of the low-flow season was required regardless of his progress on construction of the spillway.

Diversion and cofferdamming of the main river was started on July 22, 1954, when the diversion tunnel plug was blown out. This permitted the river to flow through the diversion tunnel into the forebay channel and then back to the main river channel through the auxiliary spillway.

The river was closed by first constructing the initial rockfill section of the downstream cofferdam and then constructing the upstream cofferdam in slack water. The downstream cofferdam was placed as close to the discharge



AUXILIARY SPILLWAY–TRANSVERSE SECTION
FIGURE 6

through the auxiliary spillway as feasible in order to provide a generous cofferdam cross-section. This permitted traffic in both directions on a road on the crest and upstream slope, with a minimum grade on the haul road and maximum working room in the unwatered area.

Previously, much preparatory work had been performed. Haul roads were enlarged and a temporary bridge constructed across the channel on the downstream side of the auxiliary spillway. Spoil from previous excavation, which had been wasted downstream from the powerhouse area on the left (south) bank was used for initial closure of the downstream cofferdam. Large stones and boulders weighing up to fifteen tons were sorted from this spoil pile, drilled through, and threaded with cable for easier handling.

The fill was initially pushed out from the left (south) bank until it reached a point about halfway across. Some of this was accomplished before the flood had fully receded. At this time the north bank began to scour and was then protected by placing cabled boulders, using a six-cubic yard Manitowoc crane equipped with a 145-foot boom to reach across the remaining channel. Similar boulders were also used to build up a point on the north bank. Some of the boulders were tied to the bank by cables and served as anchors until a sufficient number were in place. Others were placed in the channel bottom to form a broad crested weir and to prevent scour. The fill material used for initial closure contained practically no fines, thus permitting leakage through the fill. This operation of placing boulders and gradually bulldozing out fill material from the south bank resulted in closure on July 31, 1954. Fig. 7 shows a view prior to final closure.

This closure was accomplished with a river flow of about 14,500 cfs. Benefits were obtained by reducing releases at the Albeni Falls Dam to 7000 cfs; however, a minimum flow of 14,500 cfs was recorded during closure because of channel storage and since the closure was accomplished in less than a day. At closure the water elevation upstream from the cofferdam was El. 1995. After closure the cofferdam was immediately widened and sealed on the upstream side using bank-run sand and gravel containing some silts and clays and raised to El. 2015.

These same bank-run sand and gravels were used for the upstream cofferdam; however, this operation was much less difficult because of slack water conditions. The finer material was placed toward the upstream face and coarser material placed toward the downstream face. A relatively impervious blanket was also placed on the upstream face of this cofferdam, and then both faces were covered with broken rock riprap. The crest of this structure was built to El. 2028.

Later, a temporary line of steel sheeting was driven between the canyon walls upstream and downstream from the space to be occupied by the spillway. Sumps were installed upstream and downstream from the main spillway area with a total pump capacity of about 120,000 gpm. Difficulties were experienced in unwatering the area, principally due to leakage under the downstream fill cofferdam; however, excavation proceeded above the water line as the area was gradually unwatered.

Upon completion of the spillway and installation of the gates, both cofferdams were removed by dragline, largely from slack water. A portion of the larger boulders in the downstream cofferdam was not removed in order to further reinforce the existing boulder bed on the bottom of the river in the area immediately downstream from the spillway, thus providing scour protection.

COFFERDAM CLOSURE
FIGURE 7

Prior to this time, a section or plug of rock had not been completely excavated between the south spillway pier and the diversion tunnel in order to contain the diverted flow in the forebay channel. Upon removal of the river cofferdams, this rock was removed so that with the spillway gates in place, water would flow from the main river channel, at the spillway, directly into the forebay channel.

Main Spillway

The main spillway is a very unusual structure in that the weir slab and piers are supported by a concrete arch spanning the river between the canyon walls, transverse to the flow of the river and principal loads (see Figs. 8 and 9). Hard rock supports the structure, which spans the soft foundation.

Support of the spillway on an arch is in many respects similar to the support of a buttress of the Rodriguez Dam constructed about 28 years ago in Mexico. The principal difference is that the Rodriguez Dam is a high head structure with a positive cutoff extending to rock, whereas the Box Canyon Spillway does not have a positive cutoff.

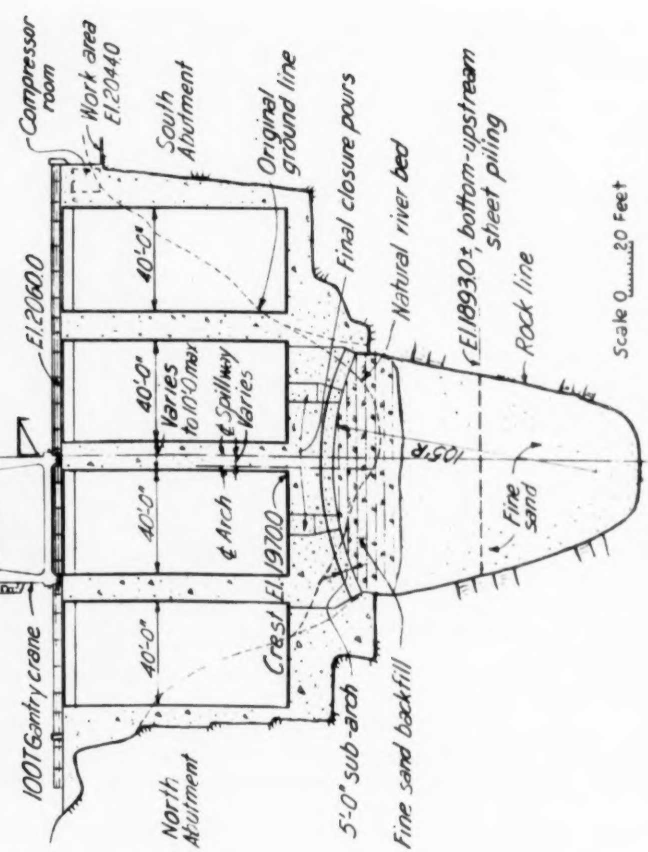
The upstream and downstream aprons are heavily reinforced concrete supported directly on the fine sand foundation. A special watertight contraction joint was installed between these aprons and the adjoining weir (arch) and the canyon wall anchorage, so that settlement can take place without damage to the joint and thus prevent loss of foundation material.

The spillway consists of four bays each forty feet wide with a crest at El. 1970. In each bay are three vertical lift wheeled gates of welded steel plate construction. The gate wheels are provided with roller bearings and are designed for a maximum load of 250,000 lbs. Each gate leaf is 20'8" high, except that in one bay a leaf has been split into two 10'4"-high leaves for sluicing trash and ice and minor regulation. In addition to the service slot, an emergency slot and a storage slot have been provided in each bay. The gate leaves can be dogged at several elevations for control of releases through a wide range of reservoir level and discharge. A 100-ton gantry crane handles the gate leaves, with a single gate leaf being engaged by separate lifting blocks at each end of the leaf.

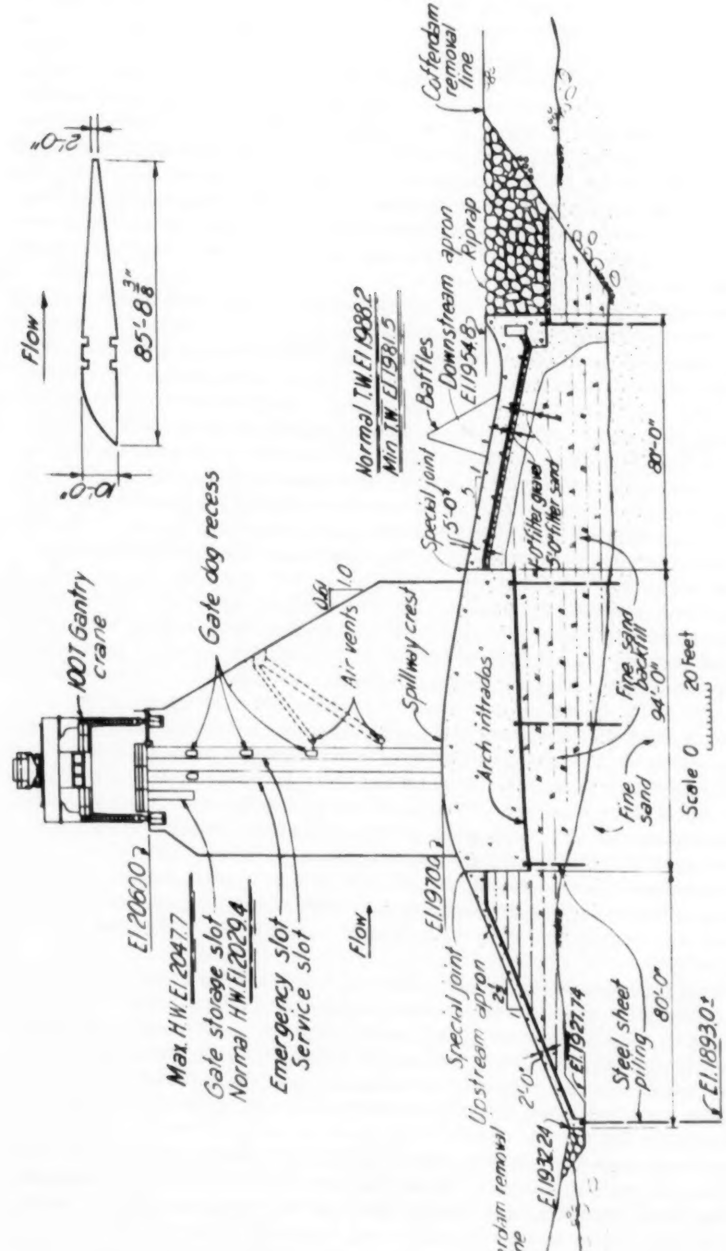
After installation it was found necessary to modify the gate dog operating mechanisms to insure positive operation rather than depend on the gravity arrangement. Subsequent operation has also shown that on partially raising the lower gate leaves sufficient turbulence is created to deposit some bed load material in the gate dog pockets and on the horizontal members of the gate leaves.

The spillway is designed to pass a flow of 350,000 cfs and has the same capacity as the upstream Albeni Falls spillway. Peak flows can be passed with hydraulic losses less than those existing in the natural channel. Although the crest of the weir is higher than the natural river bottom, the end bays (Nos. 1 and 4) were excavated out of the canyon walls so that the net width of the spillway is greater than the natural channel width. The natural approach channel was obstructed by large projections of rock from the right bank and removal of these projections assisted in reducing losses through this section.

Hydraulic model tests were performed using a fixed bed model with a scale of 1 to 60. The model included a section of the river canyon upstream from the spillway location and extended downstream beyond the powerhouse.



MAIN SPILLWAY–LONGITUDINAL SECTION
FIGURE 8



MAIN SPILLWAY TRANSVERSE SECTION

FIGURE 9

Initially, the model was calibrated to reproduce all recorded river profiles up to the record flood of 171,300 cfs and for computed profiles of 248,000 cfs and 350,000 cfs.

Upon completion of calibration of the model, the spillway, diversion tunnel, forebay channel, powerhouse, and auxiliary spillway were added to the model. Tests showed a reduction in water surface profiles upstream from the spillway. Refinements in the proposed design were then developed for the shape of the pier and apron sill, and for the extent of removal of rock projections. The basic streamlined crest shape performed as anticipated. A symmetrically shaped pier had been proposed, but the direction and velocity of approach required the adoption of an asymmetrical shape in order to minimize unbalanced forces on the 90-foot high piers.

The originally proposed design of the downstream apron was unsatisfactory with undesirable scour downstream from the structure. After some experimenting, it was decided to install baffles on the downstream apron which reduced bottom velocities sufficiently to prevent scour of the heavy riprap and natural boulder bed downstream from the apron. Qualitative tests were made of the downstream scour effects with various positions of the spillway gates when releasing flows during the power generation season. It was concluded that the discharge through each bay should be maintained as nearly equal as practical operations will permit. Soundings taken downstream from the spillway in 1957 show that no detrimental erosion has occurred.

Tests were also performed to check the capacity of the diversion tunnel and the entire diversion scheme.

The design of the spillway required that the area be unwatered and that the boulder layer, lying on top of the fine sand material in the bottom of the river, first be removed and the area backfilled with fine sand compacted in the dry. Due to delays in unwatering, on October 7, 1954, the area had only been unwatered to El. 1942. However, the arch abutments had been exposed and practically all river bed boulders had been removed by dragline. In order to control the leakage from under the downstream cofferdam and permit unwatering part of the area, the contractor was then given permission to backfill the area under the downstream half of the arch and under the upstream one-half of the downstream apron, dumping the material (approximately 10,000 cubic yards) in water. The middle and downstream rows of steel cutoff piling under the arch were then driven. It was concluded that a portion of the underwater fill would be suitable for permanent backfill if adequately compacted. This fill material was compacted up to a depth of 30 feet with Vibroflotation equipment.

The Vibroflotation equipment consisted of a large vibrator approximately 33 feet long, similar in many respects to a concrete vibrator. The vibrator unit, attached to a large diameter tube, was driven by a self-contained 60 hp electric motor and was handled by a crawler crane. The unit was lowered into the partially compacted backfill by means of a self-contained jet to the depth desired and then slowly withdrawn in one-foot increments with the vibrator in operation.

The high frequency vibration creates a fluid action which permits the sand particles to settle and close up the voids, with excess water and air rising to the ground surface alongside and around the unit. As compaction took place, additional backfill material was shoveled into the opening around the unit. The rate of withdrawal was determined by observing the current demand of the electric motor and was based on previous tests and experience of the operating personnel.

The original backfill had a relative density of about 25 per cent, based on driving tests and tests of undisturbed samples. Tests made on the vibrated backfill indicated that a minimum relative density of 70 per cent was obtained. Approximately 100 per cent relative density was obtained at the point of insertion of the Vibroflotation unit, with the relative density decreasing with the distance from the center of the unit. In this case, the unit was inserted in a staggered pattern approximately 7'0" on center in both directions. The minimum relative density was obtained at points equidistant from these locations. An approximate check on this increase in relative density determined from the fact that the backfill material added during the Vibroflotation operation equaled ten per cent of the volume of the compacted material.

These operations served to cut off water from the downstream cofferdam leakage and the backfill under the upstream half of the spillway was then placed in the dry. Backfill under the downstream apron was placed in a shallower depth of water and Vibrofloated with the top portion being placed in the dry. Completion of this work was followed by placement of filter materials in the downstream apron area.

As backfill was brought to grade in an area, it was immediately covered by a 3-inch sealing layer of lean concrete to preserve the grade, provide a working surface, and to support forms and reinforcing steel. This sealing layer was also placed on top of the filter material between the collector drains to prevent contamination and mixing of the graded materials.

Foundation drainage was designed on the basis of laboratory tests on the foundation material and filters. Based on flow net studies, uplift was assumed as 0.8 of the differential head on the structure at the upstream end of the upstream apron and decreasing uniformly to tailwater at the filter under the downstream apron. The design was also checked against the condition of failure of the upstream apron, permitting full uplift pressure under the upstream end of the weir or arch, and found to be safe.

As shown on Figs. 8 and 9, the rows of sheet piling were not driven to rock in the canyon bottom. Their depth is sufficient, however, to contain the foundation material and increase the seepage path so that the exit velocity of the seepage is within safe limits.

Additional rows of steel sheet piling were driven under the weir, in order to eliminate any direct seepage paths should the sand foundation settle. Consolidation and curtain grouting was carried out in both abutments, in addition to grouting of the sand foundation where the steel sheet piling terminated against the rock abutment faces.

Two layers of filter material were placed over the entire backfill area under the downstream apron, with the bottom layer similar in grading to fine aggregate for concrete and the top layer similar to coarse aggregate (1/4 to 3/4-inch). This filter drain is designed to prevent foundation sand from being carried into the collector drains that extend for the full length of the downstream apron and which are spaced fifteen feet on center. Each collector drain is half round, in section, with a perforated stainless steel plate in the bottom and is very similar to those used for the Petenwell[2] and Castle Rock Spillways. The seepage collected by these drains runs into a transverse tunnel and can be discharged just above tailwater, permitting visual observation and measurement. Piezometer pipes were installed in the piers, so

2. "Petenwell Hydroelectric Project," by E. Montford Fucik, Transactions, Vol. 117 (1952).

that uplift pressures can be measured at a number of locations under the upstream apron and arch. Settlement devices were also installed so that settlement of the sand foundation under the arch can be determined. If settlement occurs, the space can be grouted by means of grout pipes, which were installed in the piers. To date no measureable settlement has occurred. Maximum head has been placed on the structure and uplift pressures have not exceeded those assumed. The seepage quantity to date is insignificant. A typical set of pressure readings under the spillway is shown on Fig. 10.

The arch design considered the dead load of the structure, rib shortening, temperature changes, uplift and other hydraulic loads, various expected combinations of headwater and tailwater, and an earthquake acceleration of 0.1 gravity.

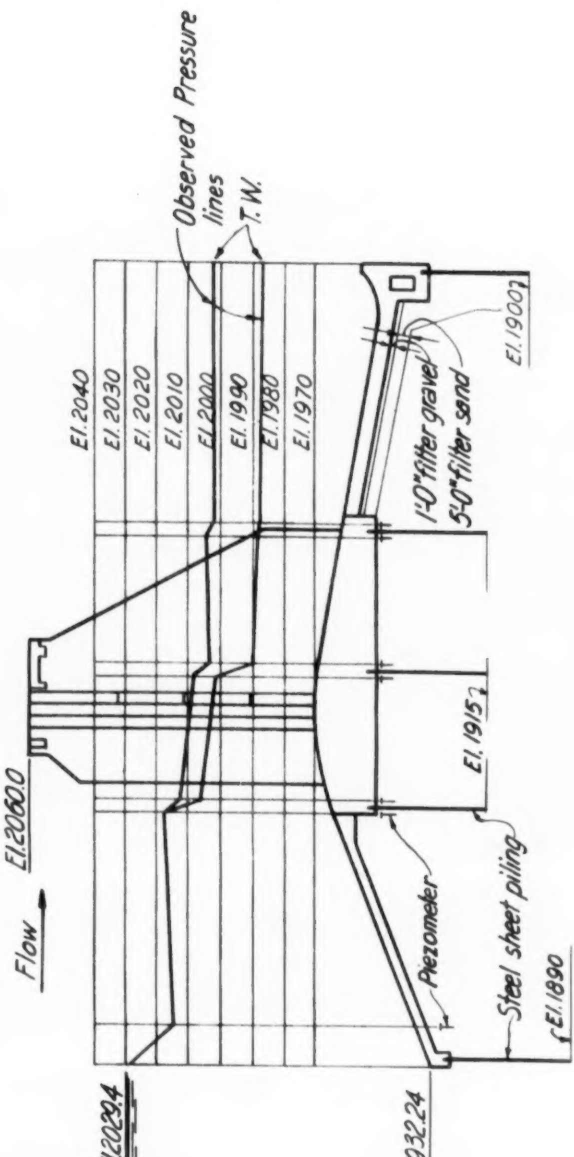
Stresses in the arch abutment are almost uniform from the upstream to the downstream edge of the arch for all conditions of loading. With the reservoir at El. 2030, the horizontal thrust on the gates and piers is counterbalanced to a great extent by the net downward load of water on the arch upstream from the gates. During flood periods, these loads are removed, together with all uplift, resulting in an almost uniform relief of stress along the arch abutments. Horizontal thrust on the piers and gates is transmitted through the arch into the abutments. The intensity of shear likewise will be almost uniform.

The arch derives its entire support from the canyon walls. The abutments were found to be of better quality rock than anticipated by exploratory drilling prior to unwatering, thus permitting reduction in the span to approximately 90 feet. In order to better fit the abutments, the arch is skewed about six degrees with respect to the centerline of the spillway. Actually only bay Nos. 2 and 3 are supported by the arch, except for a portion of bay No. 4 near the downstream end of the arch. Concreting could, therefore, be started early in bay Nos. 1 and 4, which were on the rock.

For the main arch construction, a concrete sub-arch, with a minimum thickness of five feet, was poured first. These initial pours were supported by the sand foundation, leaving small transverse closure pours to be made after the concrete had cooled sufficiently to insure that practically all shrinkage had taken place. After the closure pours had gained sufficient strength, additional pours were permitted thereon, with all dead load being taken by this sub-arch. All subsequent pours were supported by and bonded to the sub-arch and were arranged with additional closure pours. The height of the center pier was limited until the intermediate closure pours had been made. Fig. 11 is a view of the upstream side of the spillway showing the upstream apron and piers prior to completion of the deck.

A metallic aggregate topping was applied to the surface of the downstream one-half of the upstream apron, the downstream apron, and weir, in order to minimize erosion by the sand and gravel bed load of the river during flood periods.

The left (south) abutment was constructed wider than the other piers to provide space for electric lighting transformers, stationary air compressor for the air bubbler system, storage, and a spillway headwater gage well. The adjacent area between the spillway and forebay channel was preserved as a permanent work area at El. 2042. Fig. 12 is a view of the upstream side of the completed spillway, with the entrance to the forebay channel on the left. The rock plug between the main channel and forebay channel has been removed.



PRESSURE LEVELS
IN SPILLWAY SECTION
FIGURE 10

MAIN SPILLWAY DURING CONSTRUCTION
LOOKING DOWNSTREAM
FIGURE 11

MAIN SPILLWAY AFTER COMPLETION
LOOKING DOWNSTREAM
FIGURE 12

To prevent freezing and thus permit winter operation of the spillway gates, provisions have been made for side seal heaters in the piers and an air bubbler system has been provided. Provisions have also been made to heat the horizontal seals between the gate leaves. The air bubbler system consists of a header, along the edge of the deck, which supplies air to four 1/2-inch hose assemblies in each bay. Each hose extends from the deck to a point below headwater on the upstream side of the gates and is adjustable in length. The support for each hose, at the deck, is a pivoted arm which can be rotated 180 degrees. This system was adopted in preference to a submerged header equipped with orifices. If it is found necessary to pass floating ice through the spillway, the free hose ends can be quickly raised but, if not raised, it will probably not be damaged.

Up through the winter of 1957-1958 it has not been necessary to operate the spillway bubbler system or to complete and use the other provisions for winter operation of the spillway gates. During operation of the powerhouse, the flow from the main river channel is deflected upward by the upstream spillway apron and then must turn into the forebay channel. This action brings the warmer water from the river bottom up and across in front of the spillway gates and has prevented the reservoir from freezing over in this area. It is believed that only a severe winter would make the use of these facilities necessary.

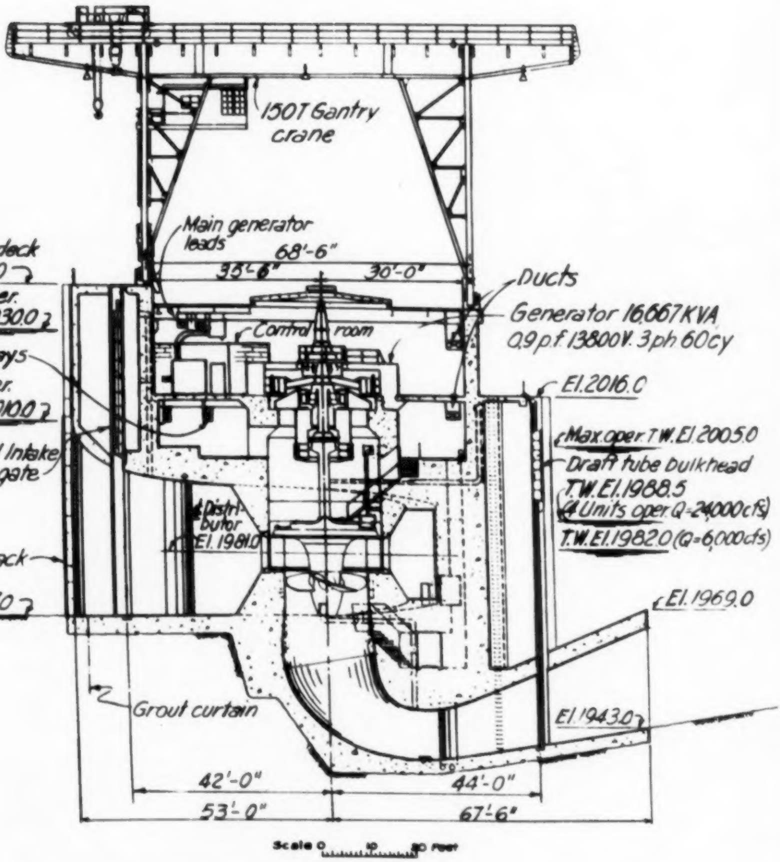
Powerhouse

The powerhouse is a semi-outdoor type of reinforced concrete construction with an integral intake (See Fig. 13). Together with the erection bay, the structure is 315 feet long. A sheet steel hatch cover is provided over each of the four generators, and two similar hatch covers are provided over the erection bay. The unit bays have both a turbine and generator floor, whereas the erection bay has a floor at the turbine floor level with an additional floor below this level. By means of an opening in the upper floor, a turbine runner and shaft can be erected without extending above the powerhouse roof level.

The rock foundation of the powerhouse proved to be of excellent quality so that foundation treatment was principally limited to a grout curtain near the upstream edge of the intake. The foundation drains are connected to tailwater and are run through the draft tube access gallery, so that the quantity and pressure of any seepage can be measured.

Each unit block was designed as an independent unit for flotation and stability, since contraction joints are provided normal to the longitudinal centerline of the units, between each of the blocks and the erection bay.

The road deck over the erection bay was designed for a live load of 1000 psf and the roof deck over the generating units for a live load of 100 psf. The 6-inch thick deck slab over the generating units is supported on steel beams and has a minimum of 0.75 per cent reinforcing steel in both directions. This heavy reinforcement, with ample pitch of the surface to prevent ponding of water, eliminates the need for any roof covering. All contraction joints are designed as watertight joints. During several years' service this deck has been watertight except for leaks along contraction joints that were accidently damaged by a rock blast during construction. These joints have since been repaired.



POWERHOUSE TRANSVERSE SECTION

FIGURE 13

The bottom two inches of the deck slab is made of concrete, using a light weight insulating aggregate. This insulation layer was poured first and contained mesh reinforcement. The stress steel was then supported on the surface of this concrete and the standard concrete poured. By means of bond between the two layers and tying of the mesh reinforcement to the stress steel, the two layers result in an insulated watertight deck of minimum thickness and weight. Heavy loads cannot be placed on this deck over the generating units and must, therefore, be handled with the gantry crane. A curb and railing separates this deck area from the erection bay deck.

The powerhouse is equipped with an outdoor type gantry crane which has a single trolley, with a main hook capacity of 150 tons and an auxiliary hook of 25 tons capacity. This crane is primarily designed for assembly and disassembly of the generating units; however, by means of cantilevering the bridge girders, this crane will also service the draft tube gates, intake gates, and trash racks. A crane equipped with two 75-ton trolleys would be preferable for some operations; however, economy dictated the selection of the single trolley.

The generator cooling water pumps and other auxiliary mechanical equipment are located on the turbine floor. Twin actuator type governors are located on the generator floor and each unit serves two turbines. The control room and main switchgear are also located on the generator floor. Cement asbestos ducts carry the generator leads just below the roof deck and underground to the switchyard located just west of the powerhouse. A 115 kv transmission line connects with the Bonneville Power Administration line two miles from the project.

CONCLUSION

This project contained a number of design and construction problems which were very unusual in character. It has been extremely gratifying to everyone concerned to observe the successful operation of the project, signifying the satisfactory solution of these problems.

ACKNOWLEDGEMENTS

The basic design was conceived and developed by the Harza Engineering Company, under the direction of the late Mr. L. F. Harza (Member, ASCE), and carried on since November, 1953 by Mr. Calvin V. Davis (Member, ASCE), and Mr. E. Montford Fucik (Member, ASCE). Field survey work, design of the switchyard, reservoir clearing, and design of the Calispel Pumping Station was performed by H. A. Sewell and J. A. Sewell (Member and Associate Member, respectively, ASCE), Consulting Engineers, Newport, Washington. Model tests were made by Dr. Lorenz G. Straub (Member, ASCE), St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, Minnesota. Construction was supervised by Mr. C. K. Willey (Member, ASCE), Resident Engineer, Harza Engineering Company. The Public Utility District was represented by Mr. V. P. Campbell (Member, AIEE), Manager. The initial phases of the construction were performed by Pacific-General-Shea, San Francisco, California, with diversion, cofferdamming of the main river channel, construction of the main spillway, and completion of other work by Morrison-Knudsen Company, Boise, Idaho, acting as construction management-consultants for the owner.

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Proceedings of the American Society of Civil Engineers

UNDERGROUND POWER PLANTS IN SCOTLAND[a]

C. M. Roberts[1]
(Proc. Paper 1675)

SYNOPSIS

The main object of the Paper is to describe the development, layout and construction of the Ceannacroc and Glenmoriston underground power stations in Inverness-shire which form part of the Moriston Scheme of the North of Scotland Hydro-Electric Board. Certain features of the headworks are of special interest and these are dealt with briefly.

The main stations and associated works are designed to produce about 206 million kWh annually from an installed capacity of 64.5 MW. The topography and geology of the sites are reviewed since these factors had a major influence on the design and layout.

Details are given of the design of surge arrangements, methods of access and ventilation, flood control, provision for the passage of fish and type of safety devices installed. Construction of the underground chambers is also described including methods of dealing with weak rock and ground water.

INTRODUCTION

The Moriston and Garry Schemes have been constructed for the North of Scotland Hydro-Electric Board as part of their general development plan in the Highlands of Scotland for power production to meet industrial and domestic load.

The two schemes are situated in adjacent glens in Inverness-shire and the various power stations are operated as one group with a control centre at Fort Augustus. (Fig. 1).

The Moriston Scheme is essentially a two-stage development utilizing the waters of the River Moriston and its various tributaries which flow into

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a. Presented at meeting of ASCE, New York, N. Y., October, 1957.

1. Partner, Sir William Halcrow & Partners, London, England.

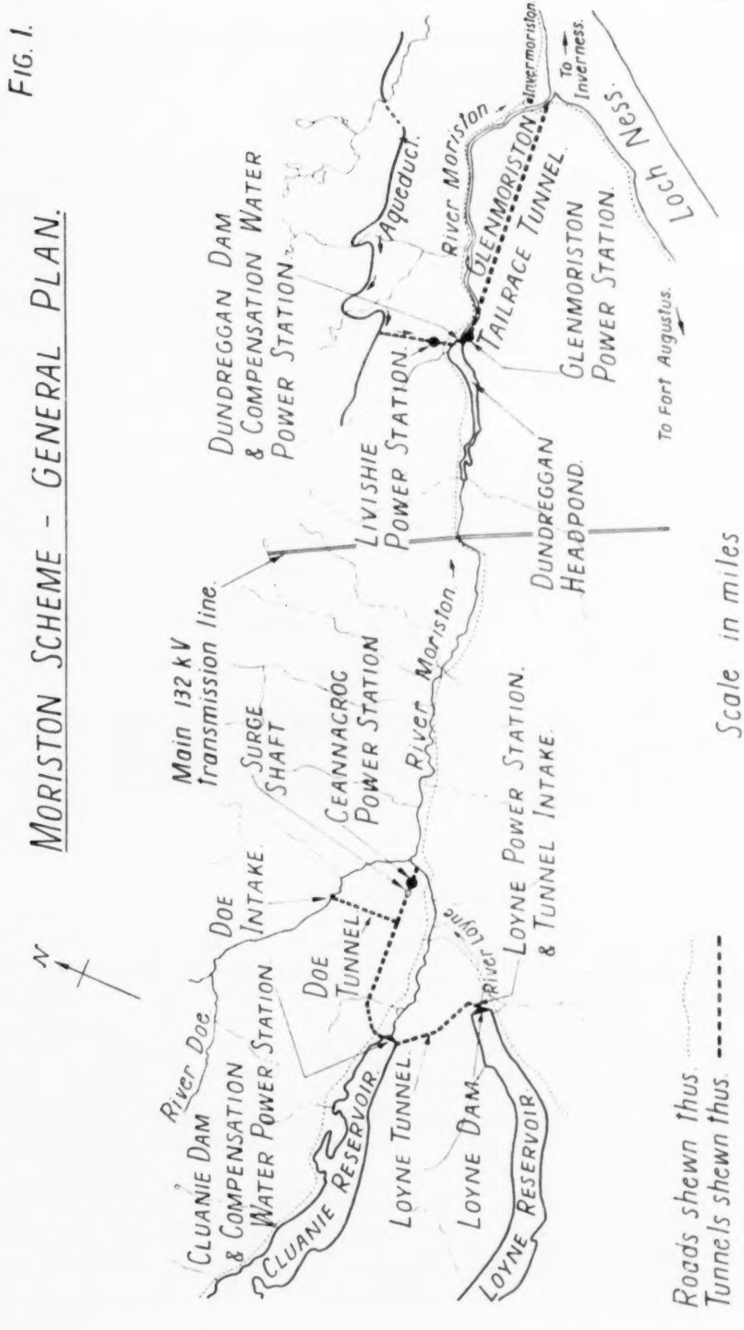


FIG. I.
MORISTON SCHEME - GENERAL PLAN.
N
CLUANIE DAM
& COMPENSATION
WATER POWER STATION
River Doe
CLUANIE RESERVOIR
LOYNE TUNNEL
LOYNE DAM
LOYNE RESERVOIR
DOE INTAKE
DOE TUNNEL
River Loyne
LOYNE POWER STATION.
& TUNNEL INTAKE.
Main 132 kV
transmission line.
SURGE SHAFT
CEANNACROC
POWER STATION
River Moriston
LIVISHIE
POWER STATION.
DUNDREGGAN
HEADPOND.
DUNDREGGAN DAM
& COMPENSATION WATER
POWER STATION.
Aqueduct.
River Moriston
GLENMORISTON
TAILRACE TUNNEL
GLENMORISTON
POWER STATION.
Invermoriston
To Inverness.
To Fort Augustus.
Loch Ness.
Scale in miles
mile 1 0 1 2 3 4 miles.
Roads shewn thus
Tunnels shewn thus

Loch Ness in the Great Glen. Regulation is provided in Lochs Loyne and Cluanie which have been raised by concrete gravity dams.

Improved methods of tunnelling which have been introduced into the United Kingdom in recent years made large underground excavations economically justifiable with the result that the main power stations at Ceannacroc and Glenmoriston have been located underground. These stations are the first two main stations in Scotland to be built wholly in this manner. Previously only one minor station was built in a deep chamber and some other stations either partly underground or in cut and cover.

The Ceannacroc station was commissioned in October 1956 and the Glenmoriston station is due to go on load in November, 1957. At a later date it is hoped to proceed with the construction of the Livishie aqueduct including a power station, also underground.

The Paper deals in some detail with the main underground power stations and the works closely associated therewith. In addition brief information is given on the regulating dams which include some novel features of construction.

Reasons for Underground Design

The adoption of underground designs for the Ceannacroc and Glenmoriston stations was governed by both technical and economic considerations. Topographical conditions were favourable for such developments. The saving of steel and cement, which are in short supply, was one of the main considerations and particularly a head development was found to offer the best results at Glenmoriston. It was also found that the adoption of unlined tailrace tunnels would effect a considerable saving in time.

Geologically the area occupied by the works is part of the Dalradian Metamorphic Complex, an area of altered sedimentary and igneous rocks. The main rocks of this geological complex found in the works area are Moine schists and granulites, grading from bands of micaceous schists to somewhat massive granulites. A wide band of horneblende schist is also present. The rock has had a close "graining" imposed on it during the process of metamorphism.

Two borings were first put down at the site of each main power station.

At Ceannacroc the cores showed undisturbed rock and this confirmed the surrounding geological indications and the information obtained as a result of the nearby tunnel excavations.

At Glenmoriston the cores showed complex jointing and some shattering. In order to obtain more detailed information a shaft was sunk on the line of the proposed intake shaft and from it a gallery was driven following the line of one of the high pressure tunnels to enter and traverse the station. The rock was shattered and jointed with the joint planes and fracture surfaces either stained red or covered with a thin green film resembling the Saponite frequently seen in the joint planes and fissures in basalt. Although the rock was in no way as good as the rock at Ceannacroc, it was considered that it would withstand the stresses imposed by the excavation and that the station could be constructed without undue difficulty.

The heads at the Ceannacroc and Glenmoriston stations are nearly the same and the flows and load factors result in an installed capacity of 20 MW and 32 MW respectively. Since the limited storage at the Glenmoriston

headpond, formed by the Dundreggan dam, made the installation of two machines desirable, three similar machines of 16 MW each were ordered achieving a considerable saving in cost and time. At Ceannacroc a 4 MW machine was installed, in addition to the 16 MW set, to be operated at times of low load and to pass the statutory compensation water.

For the Glenmoriston works three schemes were investigated:

1. An orthodox arrangement consisting of concrete lined low pressure tunnel, surge and high pressure shafts and high pressure tunnel, the latter partially steel-lined, leading to a surface station at Invermoriston.
2. A head development consisting of a concrete-lined shaft and steel-lined high pressure tunnels leading to the turbines. The tailrace was to be an unlined full flowing low pressure tunnel with the surge shaft and air-shaft just downstream of the machine chamber. Access to the chamber was to be by inclined gallery.
3. A tail development, which was similar to scheme No. 1 but with the station sited underground, and a short free-flowing tailrace tunnel and access by gallery.

Estimates of costs showed that both underground solutions resulted in a considerable saving in steel and furthermore that the head development (scheme No. 2) would save some 10,000 tons of cement although the estimated cost for schemes Nos. 2 and 3 only differed by a relatively small amount.

An additional, but important, saving was achieved by the location of the station at Dundreggan, namely the saving of approximately 4 miles of 132 kV transmission line as part of the connecting link with the Board's north-south branch of the Scottish grid. Also maintenance costs and interference with amenities were reduced.

A further point in favour of a head development was that the electrical equipment could be simplified by a combined control and switching point at the headworks.

There was a number of further points in favour of underground stations. The stations of the Moriston and Garry schemes are controlled from Fort Augustus which arrangement results in a small personnel requirement at the stations. The transformers could be sited in the open to reduce fire risk, in the case of Ceannacroc (Fig. 9) near the access tunnel portal and near the service lift shaft head at Dundreggan for Glenmoriston. These locations were suitable for naturally cooled transformers. Such an arrangement reduced the underground space requirement and simplified station layout and ventilation. No forced ventilation was considered necessary and the chambers rely on the intake of fresh air through the tailrace tunnel at Ceannacroc and through the access tunnel at Glenmoriston.

The closeness of the surge shafts to the machines and the speed and pressure regulation made it possible to dispense with relief valves.

Ceannacroc was originally planned as a surface station but the investigations for Glenmoriston and the decision to order three similar machines without relief valves led to the resiting of the generating station underground near to the surge shaft. The location of the stream intakes, which make a considerable contribution, led to the adoption of a tail development, the actual siting of the station being controlled by the position of the surge shaft. (Fig. 3). The short tailrace tunnel is free-flowing and unlined, except for a concrete invert.

Because of the relative smallness of the stations the provision of automatic flood outlets and special floor reinforcement was considered unnecessary but, in the interests of safety, the inlet valves and spiral casings were designed with a high safety factor and the turbine covers at Glenmoriston were strengthened.

The accommodation of the cables in the vertical lift shaft which is provided at Glenmoriston resulted in a further saving in the cost of electric installations. The whole arrangement whereby the control gear for the dam and intake and the switching and transformer compounds could be housed near to the future Livishie station promises further savings in cabling and operational staff.

General Description of Scheme

The Moriston Scheme utilizes the waters of a catchment area mainly above 361 ft. contour (O.D.). In the case of the upper works serving the Ceannacroc station this catchment covers an area of 101 sq. miles and in the case of the lower works serving the Glenmoriston station a total area of 155 sq. miles. Livishie has a catchment of 15.2 sq. miles.

The average annual rainfall recorded for the area of the principal works is:

Loyne	90.2 ins.
Cluanie	95.5 "
Dundreggan	81.1 "
Livishie	58.5 "

The corresponding run-off figures are:

Loyne	78.2 ins.
Cluanie	82.5 "
Dundreggan	68.1 "
Livishie	45.5 "

Further hydrological information is given in the Appendix, Table 1.

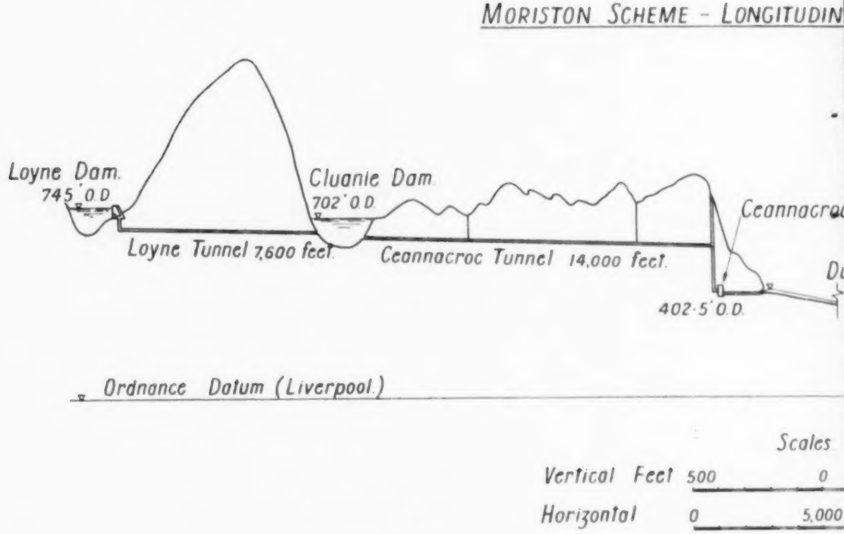
The Ceannacroc station is fed from the two reservoirs formed by the Loyne and Cluanie dams, both of mass concrete gravity section. (Figs. 1 and 2).

Loyne dam which is about 58 ft. high with a crest length of 1,745 ft. has a spillweir 225 ft. long. Two 5 ft. diameter culverts are incorporated in the dam; one fitted with a discharge regulator discharging into the headpond situated on top of the inlet shaft of the Loyne-Cluanie tunnel. The other is a dewatering culvert fitted with a jet disperser which discharges into the River Loyne. No compensation water is required from this reservoir.

To utilize the varying head differential between Loyne and Cluanie reservoirs a Kaplan 550 kW turbine running at 434 r.p.m. was installed in the foundation of Loyne dam. The turbine intake is formed by a screened square culvert through the dam and the discharge is led into the tunnel intake shaft.

The Loyne tunnel connecting Loyne and Cluanie reservoirs is horseshoe shaped of 12' 0" equivalent diameter, unlined except in sections of bad rock and is 1.44 miles long.

Cluanie dam which is 128 ft. high and 2,220 ft. along the crest is provided, in addition to the 424.5 ft. long spillweir, with scour and dewatering culverts, both of 5' 0" diameter.



A branch from the scour culvert feeds a horizontal shaft Francis compensation water turbine of 300 kW. running at 507 r.p.m.

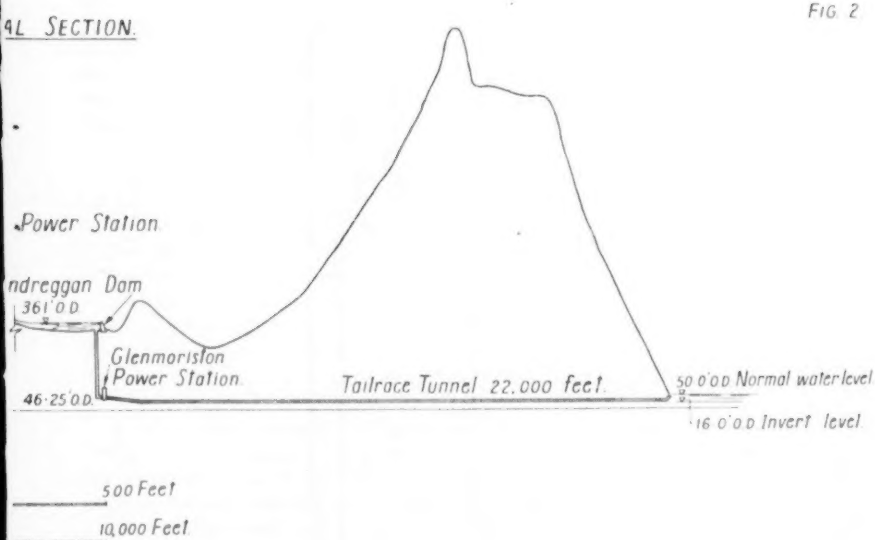
The construction of these dams has generally followed normal practice but two special features are worthy of note, namely the use of Trief cement and of pre-cast concrete panels in place of formwork.

The Trief process produced a wet ground slurry from blast furnace slag, which was obtained from Messrs. Colville's Clyde Iron Works. The slag was ground on the site of the Cluanie dam in two wet-ball mills and discharged into storage vats and from there it was pumped directly to the mixing plant serving Cluanie dam and Loyne dam and tunnel. Some was transported in circulating tankers to other mixers serving the Ceannacroc power tunnel. The slurry is added to the concrete as part of the normal weighbatching.

After some exhaustive tests on the correct mix proportions it was found that about 70% of the normally required cement could be replaced by slag resulting in a total saving of approximately 20,000 tons of cement.

In the past, work on the Scottish hydro-electric schemes has suffered from a shortage of joiners and methods which simplified or obviated the use of formwork were investigated, resulting in the adoption of precast facing slabs for both Loyne and Cluanie dams. The slabs were cast on site in special steel forms on a vibrating table and reinforcement to cater only for stresses during lifting was used. The slabs were made with ordinary Portland cement concrete.

The fabrication and setting of the slabs employed much less labour than would have been the case had ordinary formwork been used and progress was considerably increased.



From Cluanie reservoir a horseshoe shaped concrete lined low pressure tunnel of 12' 0" equivalent diameter 2.65 miles long leads to the top of the high pressure shaft, also concrete lined and of 12' 6" diameter. The bottom of this shaft is in the form of a Kaplan bend reducing the diameter towards the station to 10' 0". The short length from the end of the bend to the 16 MW turbine inlet valve is steel lined finishing with a taper of a final diameter of 6' 4". The bifurcation of the supply to the 4 MW turbine consists of a rectangular entrance opening in the north wall of the Kaplan bend. From this entrance a transition continues into a 5' 0" diameter steel lined tunnel.(Fig. 4). The high pressure shaft, transitions and upstream ends of steel pipes were all grouted in two stages in order to seal off any water-bearing fissures through which water could leak into the machine chamber and also to strengthen the rock.

The concrete linings of pressure shafts and tunnels are designed to resist the full hydrostatic external pressure at any level. Grouting of the surrounding rock after concreting of the lining ensures full supporting contact between concrete and rock and strengthens the surrounding rock to resist bursting pressure. Steel lining for high pressure tunnels is designed for full external hydrostatic pressure against buckling and this gives a thickness of steel adequate to resist internal pressure.

Two side streams are led into the low pressure tunnel, the major one, the River Doe, by means of an intake structure incorporating trash screens and a trap to exclude gravel and a tunnel of approximately 1 mile in length. The tunnel is of horseshoe section, 9' 6" equivalent diameter and concrete-lined. The Peathrain intake is similar but much smaller, the tunnel being only

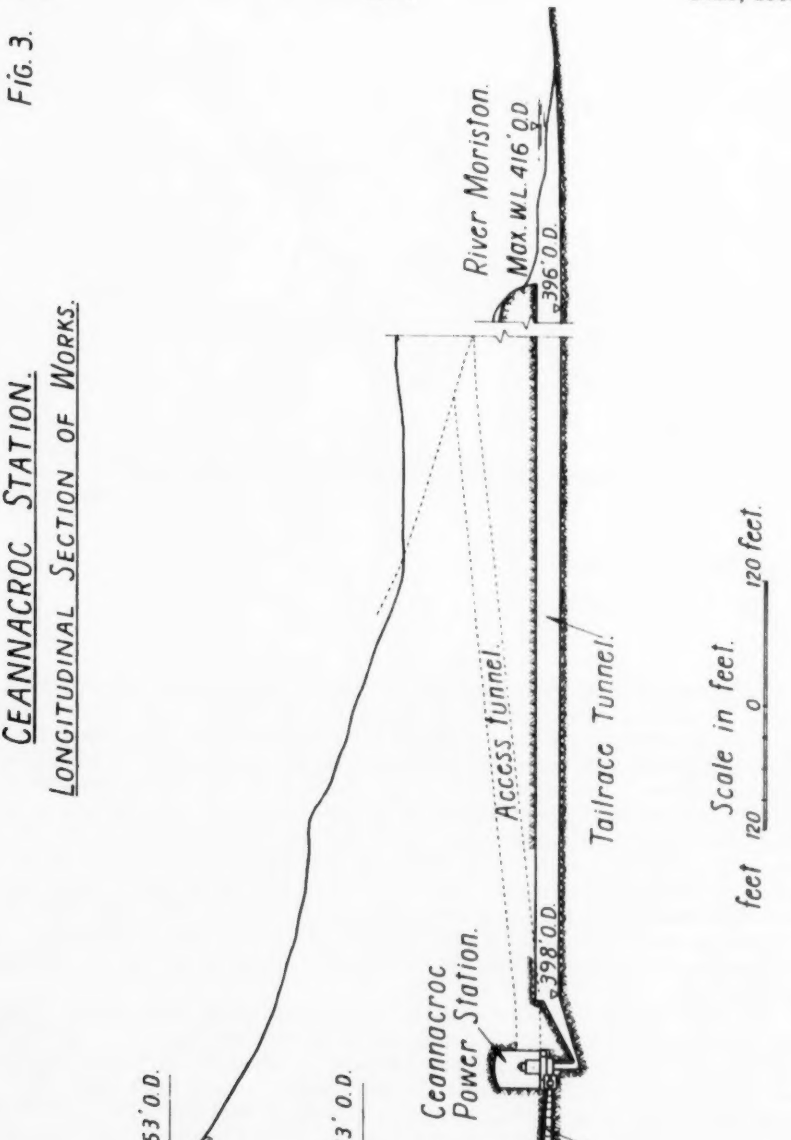

CEANNACROC STATION.
LONGITUDINAL SECTION OF WORKS.
753' O.D.
573' O.D.
Gate.
402·5' O.D.
Steel pipe.
Ceannacroc Power Station.
398' O.D.
Access tunnel.
Tailrace Tunnel.
River Moriston.
Max. W.L. 416' O.D
396' O.D.
Scale in feet.
feet 120 0 120 feet.

Fig. 3.

339' long and of 5' 10-3/4" equivalent diameter. This tunnel is also concrete-lined. Both sidestream tunnels join the main tunnel at right angles with a streamlined transition.

Dundreggan dam (Figs. 10 and 11), forming the head pond for the Glenmoriston station, is situated about 9-1/2 miles downstream from Ceannacroc. It is also of gravity section, about 400 ft. long and 55 ft. high. The main feature of this dam is the flood gates, consisting of two radial (Taintor) gates and a tilting gate. (Fig. 6). The flood control arrangements are described later in the Paper.

The dam is provided with a Borland fish pass and two culverts, one circular 42" diameter serving the compensation water turbine and one rectangular 6' 0" by 6' 0" for scour purposes. Both culverts have hydraulically operated sluice gates. The turbine of the compensation water set is of the vertical propeller type running at 1,020 r.p.m. The set has a capacity of 165 kW.

The main intake consists of a forebay formed by trash and fish screens and has mass concrete side walls, leading to the head of the pressure shaft. The piers supporting the screens are anchor-stressed and are thus of much reduced section. An emergency free roller gate is provided just upstream of the shaft.

A reinforced concrete bridge above dam crest level leads to the intake while a second bridge, constructed with prestressed precast units at a lower level downstream of the dam, provides access to the compensation water station and the transformer compound. (Fig. 6).

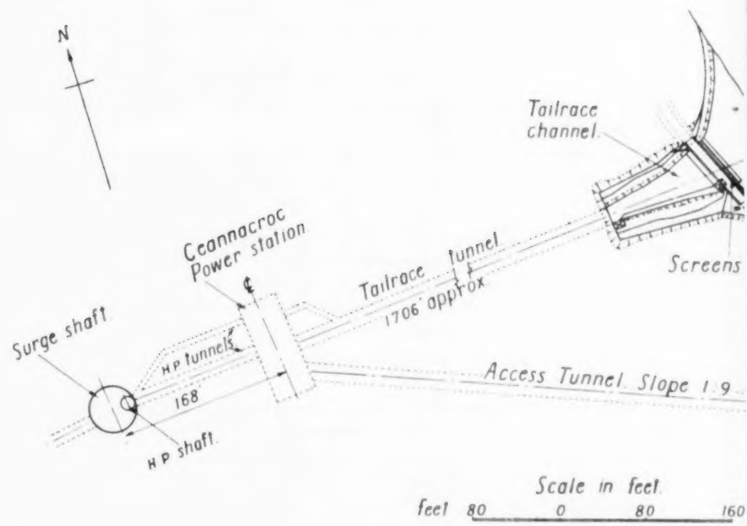
The intake shaft, approximately 296 ft. deep, is of 15' 0" diameter, concrete lined and terminating at the bottom in a Kaplan bend forming, at the same time, the bifurcation to the high pressure steel lined tunnels which are of 10' 0" diameter.

The surge shaft, of 32' 0" diameter, is situated just downstream of the turbines and from its bottom the 20' 9" equivalent diameter unlined tailrace tunnel continues for 22,000 ft. to the mouth of the River Moriston. The tailrace tunnel is concrete lined at the portal and at the surge shaft. Three methods were used to deal with sections of poor rock elsewhere in the tunnel. Concrete pillars were constructed as supports, slabby rock in the roof was strengthened with rock bolts and clay-filled fissures and "small" rock were sealed with sprayed concrete using the "Aliva" gun. The surge arrangements are described in some detail later in the Paper.

The Livishie works, which are planned for construction later, will consist of an aqueduct collecting water from several streams and bringing it to a headpond. (Fig. 1). From there the water will be conveyed by pressure shaft to an underground station with an installed capacity of approximately 11.5 MW. The tailrace tunnel about 1/2 mile long will discharge into Dundreggan headpond.

The preservation and possibly the improvement of the stock of migratory fish being an obligation on the North of Scotland Hydro-Electric Board, various measures were incorporated in the design of the works to meet this aim.

Salmon and sea trout enter Loch Ness from the North Sea and used to travel up the River Moriston to the spawning grounds situated mainly at the head of Loch Cluanie. To allow for the passage of ascending fish a compensation water flow of 65 cusecs has to be maintained in the river below Dundreggan dam from May to October and of 18.5 cusecs from November to April and this is passed partly through the compensation water turbine and

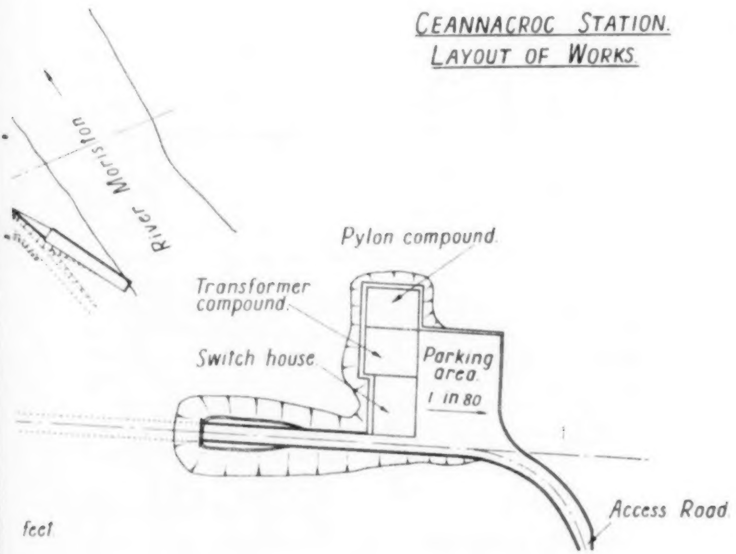


partly through a fish pass. There is also provision made for freshets amounting to 1,500 cu. ft. per year to encourage fish to ascend during the spawning season.

The outlet of the tailrace tunnel is closed by bar screens to prevent the entrance of fish attracted by the strong flow of cold water. (Fig. 6). Fish reaching the Dundreggan dam are enabled to enter the headpond by means of the Borland fish pass which operates on the principle of a canal lock and functions automatically. The entrance to the bottom of the pass is situated close to the tailrace from the compensation water station which is screened in such a manner that the line of screens forms a lead towards the fish pass. From the upper pool of the pass fish enter the headpond through a submerged opening in a gate which is fitted with a photoelectric counting device. The range of the headpond level is 4 ft. and the maximum lift required is 45 ft.

After reaching Dundreggan headpond the fish continue their travel up the River Moriston which is fed by the discharge from the turbines at Ceannacroc and several side streams. Since the raising of Loch Cluanie drowned the main spawning beds a fish pass into this latter reservoir was unnecessary but as it is undesirable that fish should reach the dam and find their way blocked a fish stopper, formed with bar screens, was constructed slightly upstream of the outlet from the Ceannacroc tailrace tunnel which is also screened against fish. This stopper is provided with a trap. Fish taken in this trap are relieved of their spawn which is transported to a nearby hatchery which has a capacity for 8 million eggs. After this artificial spawning the fish are returned to the river.

CEANNACROC STATION. FIG 4.
LAYOUT OF WORKS.



At the hatchery the eggs are stored in special tanks fed by flowing river water and shortly after hatching the salmon fry are used for restocking the river.

Arrangements for descending fish, i.e. smolts and kelts, have been made at Dundreggan and consist of fine screens at the main intake and these in turn are protected against debris, etc. by bar screens. The screens are arranged so that descending fish are led towards the upper opening of the fish pass which is the main downstream passage. There is also a kelt chute connecting into the scour culvert and both kelts and smolts can descend over the central spillweir tilting gate when it is open.

The main difficulty in operation is the cleaning of the screens, particularly of the fine smolt screens. High pressure water jets are provided to clean the latter but present methods of handling are cumbersome and there is considerable scope for improvement in this connection.

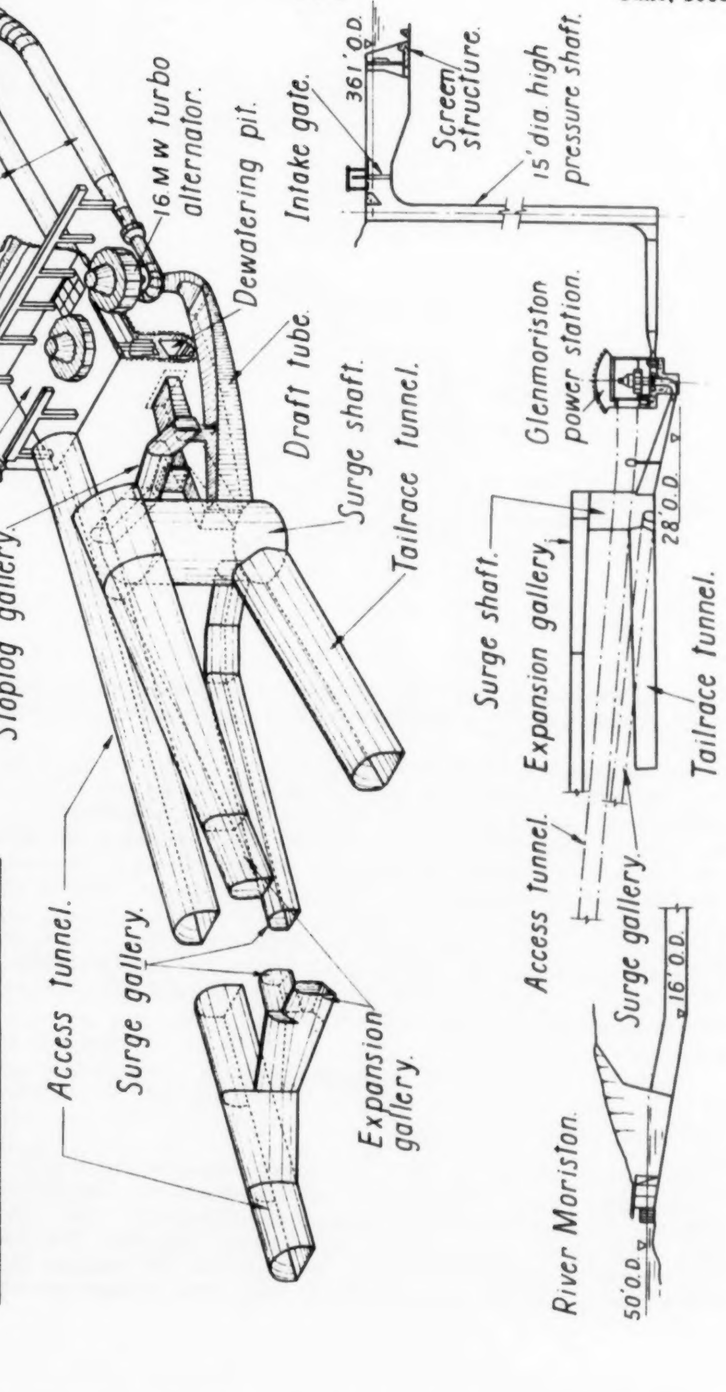
Ceannacroc Station

Generating Plant

Both the 16 MW and the 4 MW machines at Ceannacroc are of the vertical Francis type. The continuous ratings of the two sets are 18.9 MW and 4.7 MW, with 0.9 p.f. The principal machine operates between 205' and 267' net head at 375 r.p.m. with a maximum efficiency at 254' head. The small machine is capable of generating at net heads between 205' and 295' at 500 r.p.m. with a maximum efficiency at 254' head. The turbine runners are of stainless

GLENMORISTON STATION. FIG. 5.

LONGITUDINAL SECTION OF WORKS.



steel and the spiral casings of welded construction, completely fabricated in the works and delivered to site in one piece. The straight flow inlet valve for the main turbine is a spherical valve with a diameter of 6' 4", operated by external servomotors. The seals are pressure operated rubber on stainless steel seatings. The valve is fitted with an automatic by-pass valve for filling the spiral casing. The inlet valve for the smaller set is a "Roto" (cone) valve of 42 ins. diameter with Monel metal seatings.

No relief valves have been found necessary for the reasons previously stated. The alternators are provided with closed circuit ventilation and have main and pilot exciters with permanent magnet generators. The main generator has a combined thrust and guide bearing above the rotor and one guide bearing below the rotor. A further guide bearing is provided at the turbine top cover. The station auxiliaries consist of oil, drainage and de-watering pumps.

Both Ceannacroc (Fig. 12) and Glenmoriston are two-floor stations with the alternators above main floor level which is continuous with the erection bay. The lower floor gives access to the turbines, inlet valves and station drainage pits. The machine controls are located at alternator floor level. (Fig. 7).

The fire protection of the alternators consists of hydrants fed from the pressure pipes through reducing valves and the same source is used for domestic services. Special chemical extinguishers are provided for the crane.

Surge Arrangements

At Ceannacroc the surge arrangements are of a fairly simple nature. The unlined tailrace tunnel of 19' 4-3/4" equivalent diameter and 1,706' in length is designed to be free flowing at all times except following fast load acceptance coinciding with a high flow in the river. Since the reservoirs allow control of the river flow this possibility is rather remote and therefore savings in the size of the tunnel could be made and no special provisions for downstream surges were considered necessary.

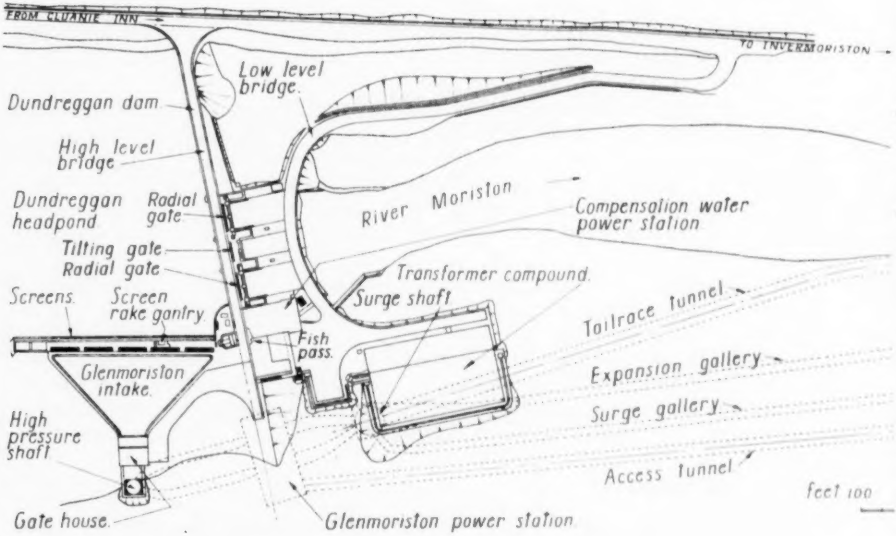
The simple surge shaft of 45' diameter and 164' depth is situated approximately 100' upstream of the station and is open to atmosphere. (Fig. 3). The concrete lining is 8" thick and provided with vents to relieve external pressure. For the protection of the underground station an emergency gate was installed at the foot of the surge shaft to close the low-pressure tunnel. This gate is electrically operated with control from the station.

Machine Chamber

The rock at Ceannacroc was generally satisfactory for the construction of an underground chamber but the full advantage of the underground design was somewhat reduced as a tail development had to be adopted.

The machine chamber which is 96' long, 47' wide and 82' high has a barrel vault roof of concrete but the sides are bare rock panelled in appearance by the reinforced concrete columns and beams for the overhead travelling crane. (Fig. 7). The roof is designed as a mass concrete arch capable of carrying a load equivalent to fifteen feet thickness of rock and this was considered adequate for any grouting pressures to be used.

The chamber was excavated by continuing the drive of the access tunnel across the loading bay to the southwest corner. Stopes were then raised up to the roof at each end of the bay and the roof excavated by means of a longitudinal heading and subsequent removal of the dumpling to 5' 0" below the arch



springing level. Next a central heading was driven at generator floor level from the loading bay for the full length of the station. This was followed by the concreting and grouting of the roof. Finally the bulk excavation of the chamber was carried out together with the necessary trimming and scaling.

The rock was generally found to be stable and sound enough to be left unlined with the exception of the south gable wall where some overhanging rock on a slip plane over the loading bay called for concreting. This was done after driving long bolts into the rock.

At the north wall some "small" rock was encountered and, as the control units were placed adjacent to this wall, it was decided to provide some protection from falling rock fragments. Chain link wire mesh was pinned to the rock by grouting in 3/8" dia. bars and bending them over the mesh. The whole surface was then gunited.

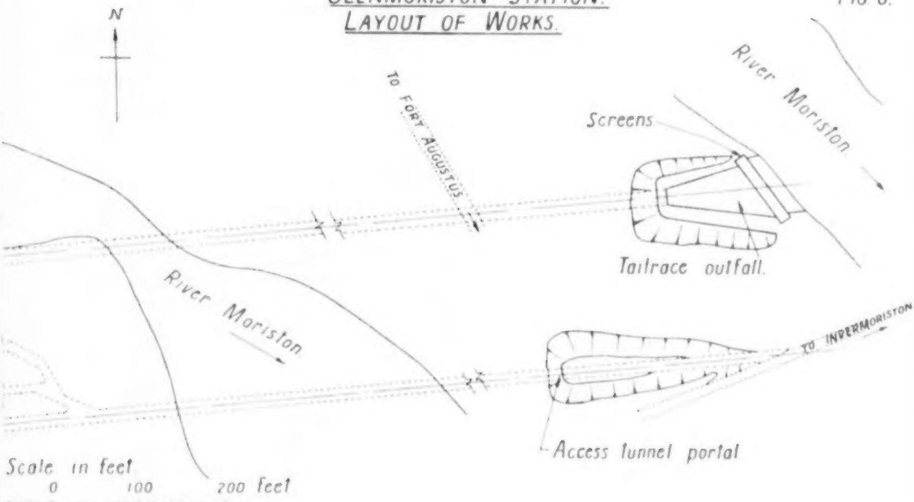
Natural ventilation is provided by fresh air entering the chamber by way of a shuttered gallery connected with the free-flowing tailrace. Hot air is exhausted up the access tunnel.

The access tunnel which is unlined except at the station end and at the portal is 620' long and is horseshoe shaped. It is 16' 0" high and 17' 4" wide and has a slope of 1 in 9. (Fig. 4).

The lighting of the station is somewhat novel and consists of continuous fluorescent trough fittings mounted behind the crane beams, allowing light to flood upwards and downwards, the effect being to overemphasize the natural rock walls and also to light the concrete roof from which, in turn, light is reflected thereby illuminating the hall. The roof, crane, crane beams and columns are painted white whilst the south gable wall is painted dull red. The

GLENMORISTON STATION.
LAYOUT OF WORKS.

FIG 6.



machine and control panels are finished in light blue. The general effect of the lighting and colour arrangement is similar to daylight.

The only heating provided is by tubular heaters in some switchgear and relay rooms.

Between the machine chamber and the switch gear, located on the surface, the 11 kV power cables and the control cables are laid in concrete ducts running on both sides of the roadway in the access tunnel.

Glenmoriston Station

Generating Plant

The station accommodates two vertical shaft Francis turbines of 16 MW each. The maximum efficiency is obtained at a head of 285 ft. As at Ceannacroc there is no relief valve and in most other respects the installations are very similar to the upper station.

Surge Arrangements

Owing to the proximity of the station to the headpond there is no surge problem upstream of the machines. Provisions for surging have been made on the downstream side and these are described below. There is no possibility of unstable oscillations occurring between the upstream and downstream systems as the headpond capacity gives more than adequate damping effect.

The main element of the downstream surge arrangement is a vertical circular shaft of 32 ft. diameter rising from 47 ft. O.D. to 123 ft. O.D. The

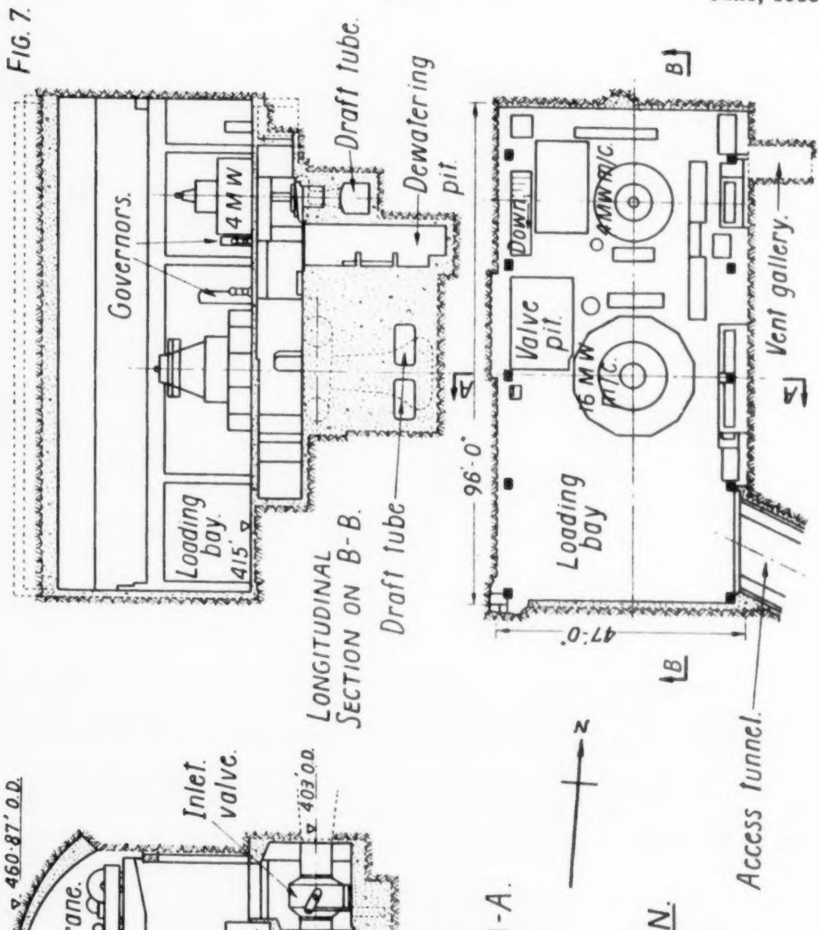


FIG. 7.

DETAILS OF CEANNACROC STATION.

effective cross-sectional area of this shaft is enhanced by a gallery, known as the surge gallery, which branches from the base of the shaft and rises at an inclination of 1 in 8.96 to a level of 123 ft. O.D. (Figs. 5 and 6).

In order to limit the maximum upsurge an expansion gallery of 12 ft. equivalent diameter is provided from an enlargement at the top of the shaft. It extends for a length of 660 ft. and is arranged to intersect the surge gallery. A short heading connects the junction of the two galleries with the station access tunnel.

This heading serves both to vent the surge system to atmosphere and also to provide a means of access, via the surge gallery, to the upstream end of the tailrace tunnel for construction and maintenance purposes. A removable bulkhead is provided to close off the lower part of the connecting heading and thus to prevent overspill from the surge system into the access gallery.

The tailrace tunnel leads off from the base of the surge shaft opposite the draft tubes and for the initial length of 1,350 ft. has a downgrade of 1 in 50.

This final form of the surge system was reached after extensive investigation by model test which was checked by analysis. The main features of interest arising from the investigations were:

(a) On rapid shut-down of the machines under fault conditions the water surface will be drawn down in the tailrace tunnel below the invert of the surge shaft. However, the steep grade of the initial length ensures that the main portion of the tunnel is always under pressure. There is no apparent tendency for sudden release of air back into the surge shaft.

(b) Under all conditions and combinations of load acceptance or rejection there will be a free water surface at the top of the surge shaft. This will ensure that the machines will not be directly subjected to shock loads should any sudden load variation occur.

(c) During the model tests it was noticed that there was a tendency for U-tube oscillations to arise between the surge shaft and the surge gallery. Subsequent theoretical analysis established the possibility of such finite but stable oscillations. It is expected that in practice natural damping factors, which could not be accounted for in the calculations or model tests, will reduce these oscillations to negligible proportions. However, in case they give rise to difficulty in governing, provision has been made for the installation of an orifice at the lower end of the surge gallery. This will provide the required damping effect on the oscillations but will, of course, increase the height of the surges in the main shaft.

The surge shaft has a concrete lining 12 inches thick. The surge gallery is also concrete lined but the expansion gallery, being flooded only under transient conditions, has been left unlined. The rock surrounding the surge shaft and gallery is grouted to reduce the seepage of water back to the machine chamber.

Machine Chamber

During the excavation of the chamber which is 104' long, 46' wide and 80' high, due to the weak and shattered rock, there was an inflow of water of about 50,000 g.p.h. and therefore after completion of the excavation it was decided that the walls would require additional support by rock bolting, lining with concrete and grouting. "Tentor" rock bolts, 9-ft. long, were grouted in with

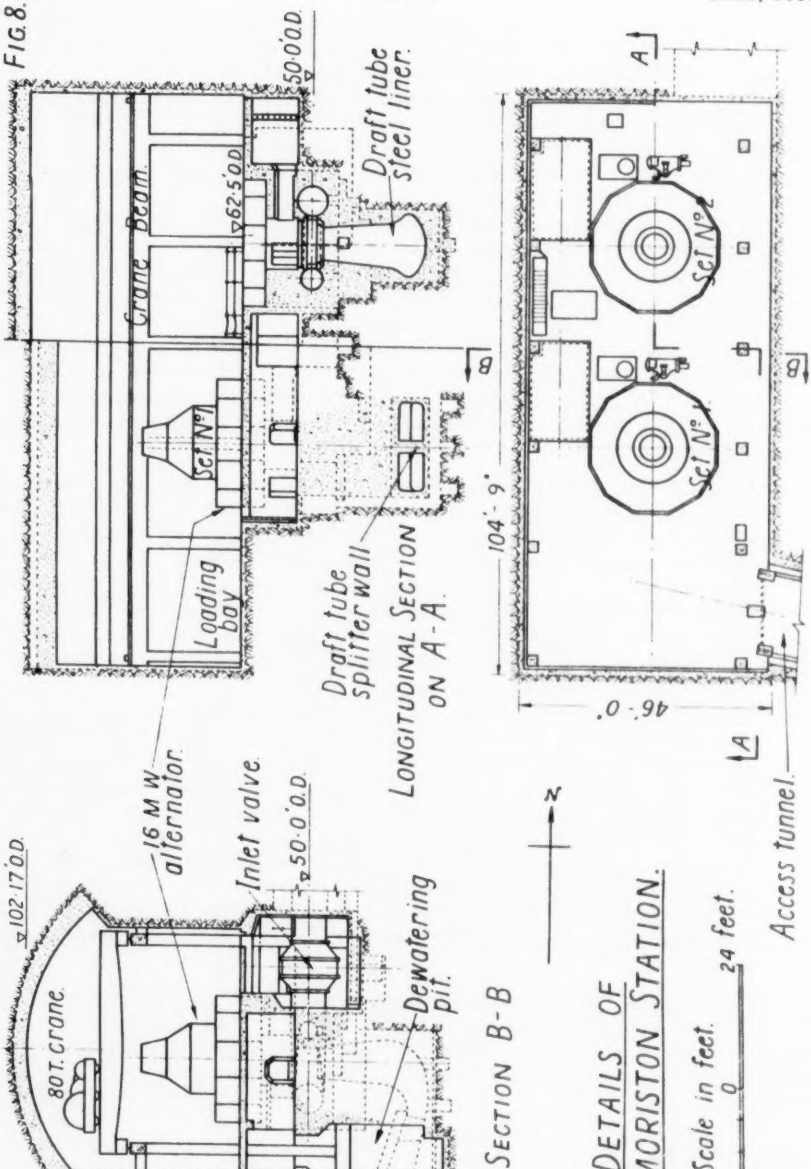


FIG. 8.
50·0' O.D.
Draft tube steel liner.
Crane Beam.
62·5' O.D.
Set No. 1
Set No. 2
Loading bay
Draft tube splitter wall
LONGITUDINAL SECTION ON A-A.
104'-9"
46'-0"
A
B
Access tunnel.
102·17' O.D.
80 T. crane.
16 M W alternator.
Inlet valve.
50·0' O.D.
Dewatering pit.
Access tunnel.
Draft tube.
SECTION B-B
N
DETAILS OF GLENMORISTON STATION.
Scale in feet.
feet 24 0 24 feet.

"Perfo" tubes at 4-ft. centres both ways on all rock walls. The "Perfo" tubes consist of perforated tubes, halved on a diameter, which are filled with mortar before insertion into the drilled holes. An indented bar is then driven into the tube thereby extruding the mortar out of the tube and filling the space of the hole. Experiments were carried out which showed that 1" dia. high tensile steel bolts in the side walls withstood a direct pull of 17-18 tons 3 days after being driven. After the bolting a concrete lining of 12" minimum thickness was applied. Reinforcement to the walls, consisting of 3/4" dia. bars at 12" centres both ways, was fixed to the rock bolts by means of special connectors screwed to the threaded ends of the bolts. The walls were subsequently pressure grouted in three stages and finally shallow pressure relief holes were provided.

The grouting scheme adopted was as follows:

1st Stage:	8' deep holes 4'6" centres both ways, grouted at 20 lb/sq. in.
2nd Stage:	redrilled to 12' depth, 50 lb/ sq. in.
3rd Stage:	redrilled to 20' depth, 100 lb/sq. in.

During grouting some holes were left open to act as relief holes.

The effective life of rock bolts is still uncertain and it is considered that the grouting of the rock, apart from the sealing of water bearing fissures, contributes towards the permanent strengthening and stabilizing of the walls.

Also attempts were made to relieve water bearing fissures by drilling long holes from the surge shaft over the chamber and this has shown good results in reducing inflow to the chamber. This drilling continues at the time of writing.

The concrete roof is of the same construction as at Ceannacroc but in order to protect the machine erection from dripping water it was found necessary to construct a temporary lining of corrugated sheeting. A further stage of grouting of the roof will be carried out later when plant installation allows access. Provision has been made to construct, at a later date, a false ceiling should the grouting of the roof not entirely eliminate leakage.

The overhead crane was needed at an early stage to be available for plant erection and it was necessary to design the crane columns for a longer length than that finally required. In an attempt to reduce this waste a design for supporting the crane on the roof springing was developed but unfortunately the rock was not sound enough to carry the load. Crane columns and beams are of fabricated steel sections and encased in concrete.

Connected by a short passage to the access tunnel is a separate stop-log gallery running across the draft tubes. (Fig. 5). The stop-logs are steel bulk-heads operated by hand hoist running on a monorail and are used to close the draft tubes when dewatering of the machines becomes necessary. The closure at the upstream end is effected by the inlet valves, which are of the same type as that fitted to the larger machine at Ceannacroc.

The ventilation consists of the escape of warm air from near the crown of the station roof, this being exhausted through trunking in the lift shaft. Cold air can enter the chamber through the access tunnel.

The access tunnel is 2,200 ft. long and of the same section as at Ceannacroc sloping at 1 in 12. The shaft accommodates a passenger and goods lift, main and control cables and ventilation trunking.

Fig. 9. Ceannacroc Station—Access road, switching and transformer compound.

Flood Control

The conditions imposed by the parliamentary powers at Dundreggan dam restricted the permissible rise in water level to 1 ft. above the "crest" when the river flow was equal to a "normal maximum" flood of 15,000 cusecs and to 3 ft. above the crest when the flow was equal to a catastrophic flood of 30,000 cusecs. There was, of course, no difficulty in providing flood openings through the dam, controlled by gates, adequate to pass these flows without exceeding the specified levels. The difficulty was in devising a system, which had to be automatic, for operating the gates in such a way that:

(a) The level would remain as near crest level as possible in normal times to give the maximum possible operating head at the turbines.
(b) As little water as possible would be wasted in discharging water in a rising flow which does not develop into a sizeable flood.
(c) The outflow would vary at about the same rate as the natural flow to avoid undue disturbances to riparian owners.
(d) The minimum gate movement would be called for in order to conserve the charge in the batteries which would supply current.

The problem was complicated by the fact that a sudden rejection of load, or 2,000 cusecs, by the Glenmoriston machines would cause a rapid rise in headpond level which would be "interpreted" by an automatic system as the onset of a flood and the gates operated accordingly. In addition account had to be taken of the fact that the proposed future Livishie generating station would discharge directly into the same headpond and a sudden acceptance of load at that station would have an effect cumulative to rejection at Glenmoriston; also acceptance at Ceannacroc, 9-1/2 miles upstream, causes a flow surge which is still appreciable at Dundreggan.

The means of flood discharge were chosen as one 2,000-cusec tilting gate, selected because a free level discharge opening was desirable to deal with trash and descending fish, and two 14,000-cusec radial gates because, for a discharge of this magnitude, they were the most economical. (Fig. 6).

For the automatic control various systems were considered. First a separate system was adopted for the tilting gate such that it would be linked directly with the Glenmoriston machines so that any change in water consumption by the machines would be immediately compensated by a suitable opening or closure of the gate; any rejection or acceptance by these machines would then not cause any sudden change in headpond level, leaving only the much less pronounced changes due to the other stations and the natural changes in flow to be dealt with by the two radial gates. A study was made of hydrographs recorded over 25 years mainly to determine the maximum rate of increase in flow to be catered for. On the result of this study it was determined that the most severe condition which had to be catered for was the possibility of a flood commencing while the headpond was drawn down to its lowest operating level of 257' O.D. Then while the level rises the flow would increase so that when the water reaches the level which operates the gates a flood flow of 6,000 cusecs may have to be passed. This condition is more severe on the control system than having to deal with the full catastrophic flood of 30,000 cusecs. This consideration led to the adoption of a speed of gate movement corresponding to 700 cusecs per minute.

As for the systems of operation of the gates, various systems were considered including one linking directly the gate opening to the inflow

Fig. 10. Dundreggan dam—Downstream face.

measured at a point upstream of the headpond which was rejected because there might have been a continuing out-of-balance between inflow and outflow which could have drained out the headpond or caused an excessive rise. The choice was narrowed down to two possible: "two-level reversing" and the "multi-level" system. In the former an upper level would start the gates opening and they would go on opening until the water level had risen to a peak and dropped by a "resetting margin;" a lower level would start them closing and they would go on closing until the water had dropped to its lowest level and risen again by the resetting margin. In the "multi-level" system each time the water reached one of a dozen opening levels the gates would open to a definite setting; they would be closed to the lower setting when the water had dropped by the resetting margin below the corresponding opening level. The system of operation is still under consideration.

General

The total cost of civil engineering works and generating plant for the Upper and Lower Moriston Works was £10,000,000 ($28,000,000) giving a cost per unit of 0.615d (0.71 c).

These figures are based at a ruling labour rate of 3s.11-1/2d. per hour (55.4c) and at a rate of exchange of £1 equivalent of $2.8.

Access to sites generally presented no problems as existing main roads could be used but some bridge works and road improvements were necessary.

An access road to Loyne dam about 3 miles long had to be constructed including three Bailey bridges.

Cement was supplied from factories in Kent, transport being by sea to Inverness and from there by road to sites.

Although the country rock in this area was mica-schist, there is a granite intrusion near Cluanie dam and a quarry was opened there to provide all the aggregate for the concrete of the Upper Works. Building stone for masonry work was also obtained from the same quarry.

A camp for workmen employed at the Upper Works was built at Cluanie, about 1/4 mile downstream of the dam. The camp accommodation at peak period was 800 men, about 80 of whom were engaged on the power station.

At the Lower Works two camps were established by the Contractor, one at Dundreggan for about 200 men and one utilizing a converted shooting lodge at Livishie for about 90 men. The remainder of the men were transported to site daily. The Dundreggan camp site also contained stores, workshops and Contractor's and Resident Engineer's offices. At the peak period 350 men were employed on the works.

For the Lower Works concrete aggregates were brought by road from the Contractor's gravel pits about 30 miles away. Building stone was partly obtained locally, partly from Cluanie quarry. Precast concrete members were produced at the Contractor's affiliated precasting yard at his gravel pits.

CONCLUSIONS

Geological exploration in sufficient detail is unavoidably limited and even when the general geology is well known the area of a proposed underground chamber may well present surprises to the Engineer. Certainly two borings per chamber cannot be considered enough unless they are only required as a

Fig. 11. Dundreggan dam—upstream face with main intake.

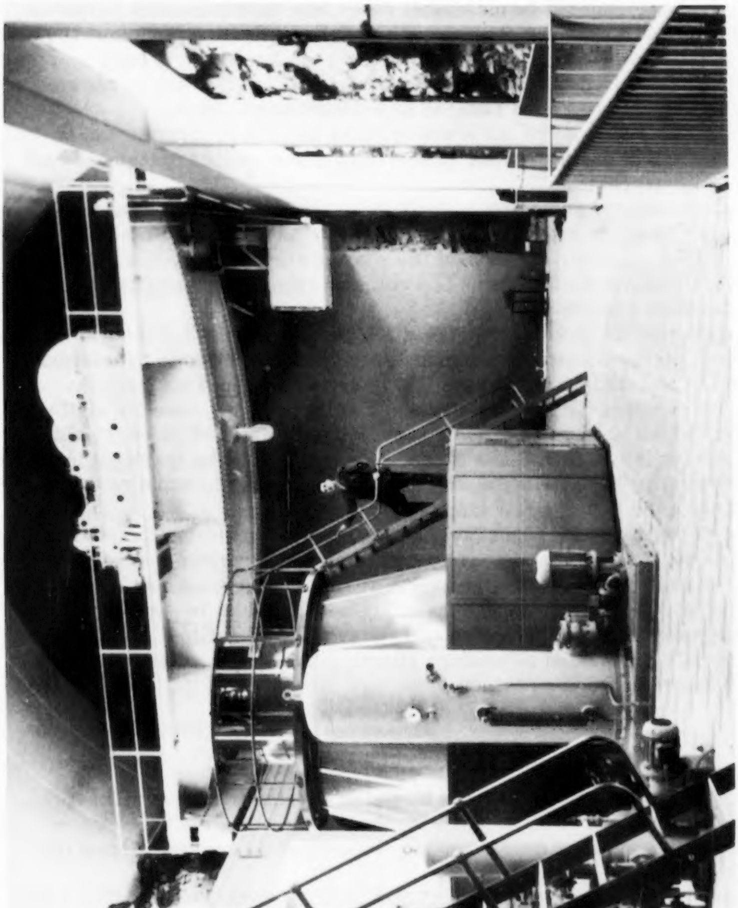


Fig. 12. Ceannacroc station—internal view.

check on known physical conditions. Particularly information on the expected ingress of water may not be obtainable other than by exploratory galleries. It would seem that with the development of the rock bolting technique the possibilities of forming large chambers in poor rock have been greatly increased and therefore the limitations imposed by poor rock or the lack of knowledge of rock conditions have been reduced to a considerable extent.

There is however, scope for investigations into the properties and behaviour of rock at depth in order to arrive at more precise designs of roof and side wall supports.

Owing to modern excavation methods and increased efficiency the cost of tunnel excavation has not risen at the same rate as the cost of other civil engineering works. Taking into account the shortage and rising prices of steel and cement, underground stations and unlined tunnels are therefore becoming more attractive economically.

In the case of the stations described the central control for the group, of which these stations form a part, made possible a reduction in operational personnel in the stations which resulted in reduced space requirements. At Ceannacroc the generating station space in relation to the capacity of plant installed works out at 505 cu. yd./MW and at Glenmoriston at 329 cu. yd/MW. The reason for the higher figure for Ceannacroc is that the leading dimensions were governed by the 16 MW set and it was not possible to make reductions in the area occupied by the 4 MW machine.

Although the space for plant erection is limited in underground stations careful planning of the construction programme can overcome these difficulties. On the other hand freedom from restrictions imposed by climatic conditions, particularly in the Scottish Highlands, results in a saving in time.

Model experiments for surge conditions were fully justified and, in the case of Glenmoriston, revealed subsidiary effects which would not otherwise have been fully appreciated.

ACKNOWLEDGMENTS

The Consulting Engineers for the Moriston Scheme were Sir William Halcrow & Partners of London, England, the Author being the partner responsible for the detailed design and supervision of the construction of the works.

The Author is indebted to the North of Scotland Hydro-Electric Board for permission to publish the information given in the Paper and desires to express his thanks to Mr. Max Rothschild and Major K. F. Fullagar for their assistance in its preparation.

TABLE 1.

RESERVOIRS AND HYDROLOGY OF THE MORISTON SCHEME

	Loyne	Cluanic	Glenmoriston	Livishie (proposed)
Useful Storage capacity				
Cubic feet x 10^6	1,600	6,090	Negligible	
In million kWh through associated stations.	18.3.	67.8.	"	
Catchment area sq.miles	26.5.	74.5.	155	15.2.
Average annual rainfall ins.	90.2.	95.5.	81.1	58.5.
Average annual run off ins.	78.2.	82.5.	68.1.	45.5.
Sillage and compensation losses cusecs	8	85	140	5
Average flow available for power cusecs	128	367	642	46
Spillweir crest ft. O.D.	745	702	361	
Maximum drawdown ft.	39	67	4	
Type of Dam	Concrete Gravity	Concrete Gravity	Concrete Gravity	
Height ft.	58	128	55	
Length ft.	1,745	2,220	400	
Vol. cu.yd.	66,000	232,000	35,000	
Length of spillweir ft.	225	424.5	77	

TABLE 2.

TUNNELS AND SHAFTS OF THE MORISTON SCHEME.

6

	Length	Diameter	Slope	Construction
Loyne Tunnel.	7,600ft.	12ft.0ins. equivalent	1 in 800	Unlined, horseshoe cross section.
Ceannacroc Tunnels.				
(low pressure)	14,000ft.	12ft.0ins. equivalent	1 in 378	Horseshoe cross section concrete lined.
(high pressure - two)	100ft.	10ft.0ins.	1 in 21.1	Two steel-lined high pressure tunnels to machines in Ceannacroc power station
	111ft.	5ft.0ins.	1 in 231	
(Tailrace tunnel)	1,706ft.	19ft.4¾ins. equivalent	1 in 853	Horseshoe section, unlined. except for invert.
(Peathrain intake tunnel)	339ft.	5ft.10¾ins. equivalent	1 in 2.5	Horseshoe cross section, concrete lined.
(Doe intake tunnel)	5,781ft.	9ft.6ins. equivalent	1 in 40.36	Horseshoe cross section, concrete lined.
(high pressure shaft)	170ft.6ins.	12ft.6ins.	Vertical	Concrete lined.
(surge shaft)	180ft.	45 ft.	Vertical	Concrete lined.
Glenmoriston Tunnel.				
(Tailrace)	22,000ft.	20ft.9ins. equivalent	1 in 50 and 1 in 5,250	Horseshoe cross section, unlined.
(high pressure shaft)	296ft.	15 ft.	Vertical	Circular section, concrete lined.
(high pressure tunnels - two)	100ft.	10ft.0ins.	1 in 100	Steel-lined.

TABLE 3.

PLANT OF THE MORISTON SCHEME

	Main Plant			Subsidiary Plant		
	Ceannacroc	Glenmoriston	Loyne	Cluanie	Dundreggan	Livishie (proposed)
Type of turbine	Vertical Francis	Vertical Francis	Vert. Kaplan	Horiz. Francis	Vertical Propeller	Not decided
Capacity	1 x 16 MW 1 x 4 MW	2 x 16MW	550kW	300kW	165kW	11,500kW
Speed of turbine r.p.m.	375 500	375	434	507	1020	-
Speed and pressure control.	Governor alone.	Governor alone.				
Average gross head ft.	276	310.5				
Design head ft.	250 254	285	39	86	42.5	826
Average annual output kWh x 10^6	60.8	120.2	1.2	2.1	0.6	21.2
Annual load factor %	40	41				

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CIVIL ENGINEERING FEATURES OF LINDEN GENERATING STATION[a]

A. Verduin,[1] A. M. ASCE
(Proc. Paper 1676)

SYNOPSIS

The object of this paper is to set forth the problems which the Civil Engineer meets and is called upon to solve to make possible the construction of an electric generating station involving the rather unusual conditions which occurred in the construction of the Linden Generating Station of Public Service Electric and Gas Company of New Jersey.

This station is situated about seven miles south of Newark on the west bank of the Arthur Kill, a navigable channel which separates New Jersey from Staten Island. At this location it adjoins the refinery of the Esso Standard Oil Company. The construction of the generating station at this location resulted in a mutually advantageous combination of two industrial processes; namely, the processing of crude oil to refined petroleum products and the conversion of refining residuals to electric energy. The contractural arrangement provides that Public Service will deliver to Esso the steam requirements of the Bayway Refinery in exchange for fuel and raw water. The additional fuel required to operate the units at full electric output is to be delivered by the refinery at commercial rates. The basic fuel for both commitments will be high viscosity residual fuel and any deficiency made up with Bunker C fuel oil. Raw water delivered by the refinery in exchange for steam will be purified, filtered and demineralized at the generating station. The exchange of these by-products has been termed the "Big Swap".

The initial capacity of the station is 450,000 kw. Each of two units has a turbine generator rated at 225,000 kw. The steam pressure and temperature of Unit No. 1 are 2000 psi and 1050 F respectively; of Unit No. 2, 2350 psi and 1100 F. Reheat steam temperature in Unit No. 2 is 1050 F. The maximum extraction steam to the refinery is 1,550,000 pounds per hour.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1676 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 3, June, 1958.

a. Presented at meeting of ASCE, New York, N. Y., October, 1957.

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Site Preparation

The station property, comprising approximately 116 acres, extends about a quarter mile along the west bank of Arthur Kill, from which the condensing water is obtained. Fig. 1 shows the layout of the station, north is to the left with Arthur Kill at the top. Running across the center of the property from left to right is the single track branch line of the Central Railroad of New Jersey. At the west side are the main shore line tracks of the railroad shown at the bottom of the map. The generating station is east of the branch line track, together with transformer yard and water treatment plant. You will also note the screen house at the intake for condensing water and the steam, oil and water pipe line starting on their mile long trek to the Standard Oil Plant. At the right is the discharge canal extending to the relocated Piles Creek which runs along the south property line. West of the branch line track are the switch yards, oil tanks and the future coal storage area. It can be seen that the major part of the construction work was between the Arthur Kill and the branch line of the Central Railroad of New Jersey.

Although the site had the advantage of nearness to the Esso property, and available condensing water, there were a number of disadvantages. The area was largely a tidal swamp, with a major portion under water at times of extreme high tide, and the nearest paved road was three-quarters of a mile away from the center of the property, with two creeks intervening. One of these creeks meandered through the section of the property at the site of the power station building (Fig. 2). Furthermore, the property was crossed by a 26-inch high pressure natural gas line and a group of four smaller gas and oil lines, and bisected by a branch line of the Central Railroad of New Jersey. One corner of the property had been filled but the fill consisted of chemical waste with high acid content from an adjacent plant, together with the remains of a demolished building consisting of huge pieces of broken concrete that had been trucked to the site for what was thought to be a final resting place.

The first task was the construction of an access road across the meadow land, together with the necessary bridges. For this access road, a total of 85,000 cubic yards of fill was required. It consisted of a mixture of dry sand and gravel and was trucked to the site from borrow pits about five miles away. The particular problem in connection with the construction of the road was to so place the fill that it would not result in the failure, with accompanying heaving of the meadow mat and organic silt which covered the area to a depth of fifteen to twenty feet. Tests of strength of this underlying material indicated that there was a strong possibility of failure unless the fill was placed with several months intervening between the deposition of the lower and the top layers. Time was not available to build up the strength of the underlying material so the fill was placed as carefully as possible and a chance taken on the possibility of failure. Fortunately, little heaving occurred.

The bridges (Figs. 3, 4) across the creek were constructed of creosoted fir timbers on creosoted pile bents located on ten foot centers. Bridge decking was creosoted oak with steel tread plates.

The first major project at the Power Station site was the filling of the area. Mean high tide at the site was about one foot below the top of the meadow mat but high high tide was about five and one-half feet above the top of the mat. It was, therefore, necessary to bring the finished grade of the property to a level above this high water line. At the same time a new channel had to be dredged along the side of the property to divert Piles Creek where it crossed

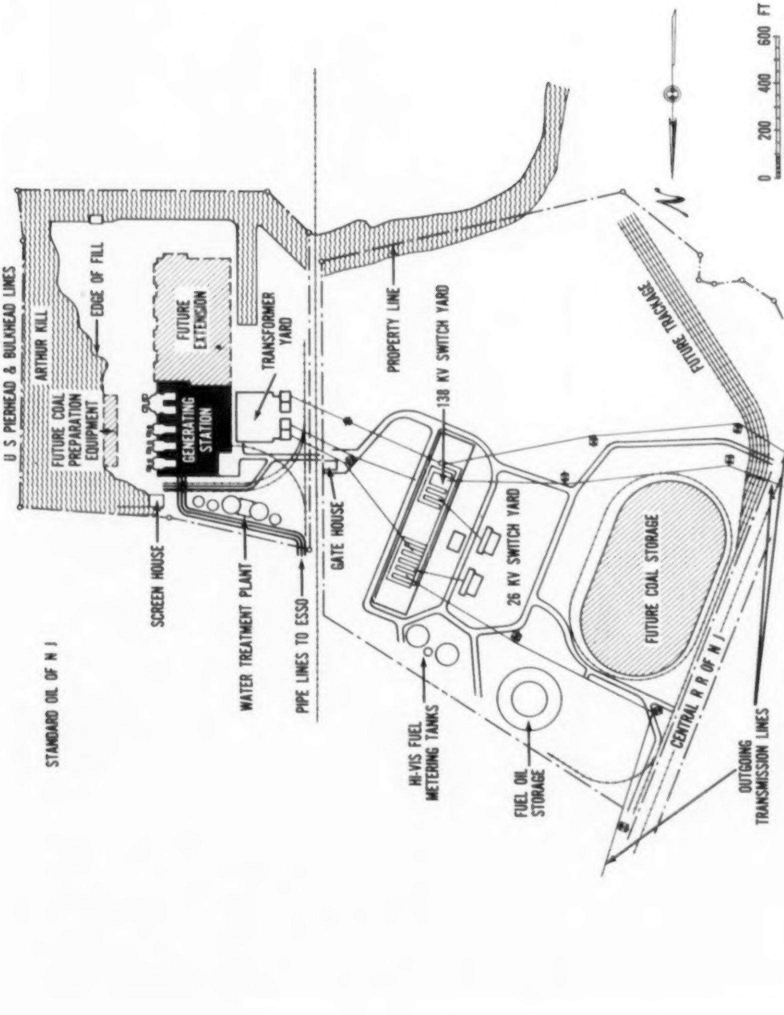


Fig. 1—Property Map

Fig. 2—Generating Station Site. Boring rig at work in Piles Creek with Arthur Kill beyond.

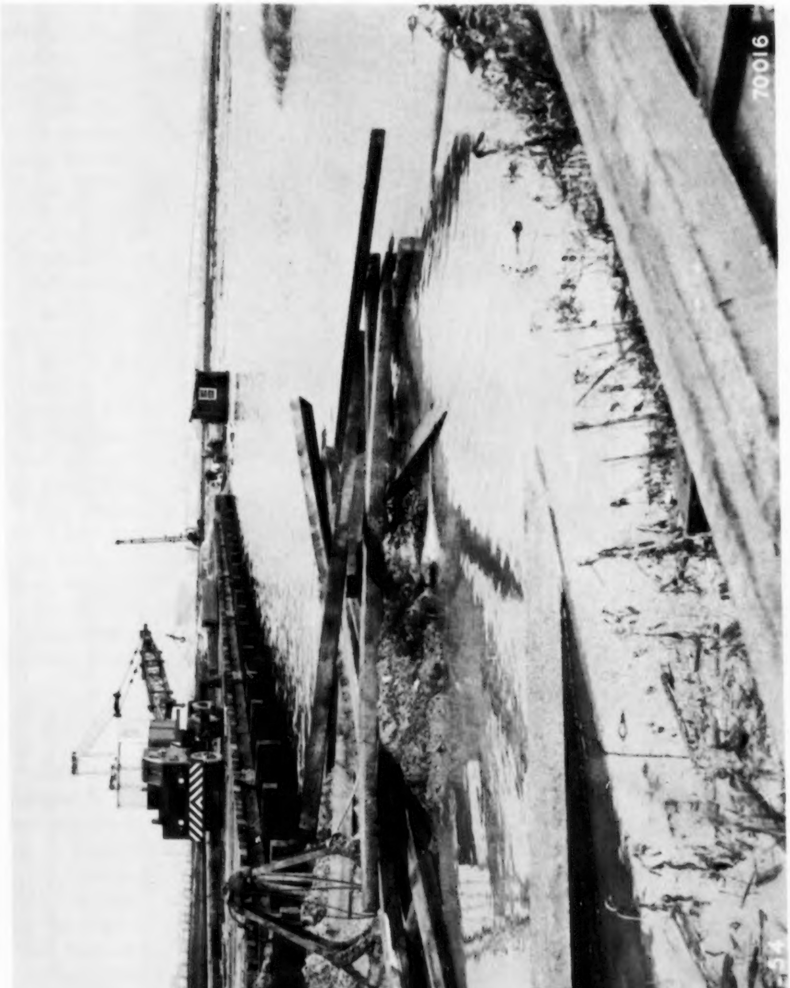


Fig. 3—Bridge over one of the creeks. Water is at high tide.

J.O. 8160 (7) Access Road and Bridge over Winans Creek - Looking Northeast
70337
9-28-54 Linden Generating Station Public Service Electric and Gas Company

Fig. 4—Access road fill and bridge over another creek. Water is at low tide.

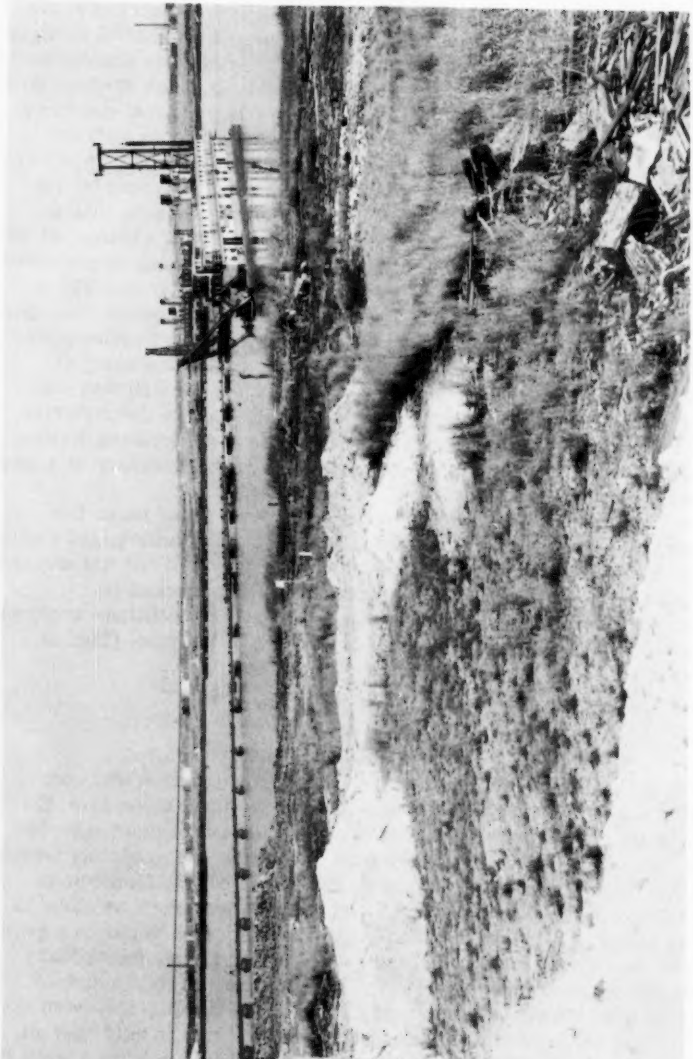
the station area, (Fig. 5) and the original creek bed filled. A total of some 900,000 cubic yards of fill was required. Unfortunately the material dredged from the channel for the creek was unsuitable for fill as well as inadequate in quantity, and Arthur Kill in front of the station had already been dredged down to rock. Furthermore, trucking of fill to the site was impractical due to the large amount to be handled and the fact that the access road was still not ready for heavy traffic. It was necessary, therefore, to find some way of securing fill via the Arthur Kill. There were three possible sources; (1) material previously removed from the Kill and deposited on a nearby island which could be pumped to the station site, (2) sea sand from the bottom of the bay which could be transported to the site in hopper dredges and then pumped onto the property, and (3) material dredged from sections of Arthur Kill a few miles south of the station, loaded on scows with dipper dredges, transported to a handling basin on the opposite side of the Kill and then pumped under the Kill to the site. Although the third scheme meant double handling, it proved to be the least costly (Fig. 6). The material from the Kill was also superior, being a coarse sand and gravel. Moreover, some of the material was dredged from in front of the Public Service Sewaren Generating Station, about six miles away, an operation which would have been necessary at a later day regardless of the work at Linden.

The material was deposited on the property to a level about three feet above the proposed final grade of the station to take care of anticipated settlement (Fig. 7). A total of approximately 640,000 cubic yards of fill was deposited on the property in this operation, the remainder being trucked in.

Work was progressing smoothly when adverse weather conditions accompanied with damaging high tide and wind caused a break in the dike, (Fig. 8) spilling water from the Kill onto the construction area.

Design and Construction

Sufficient borings were taken on the site, before construction was commenced to afford a clear picture of sub-surface conditions. In general, the borings showed from ten to twenty feet of meadow mat and organic silt over ten to fifteen feet of medium to fine gray sand with some silt gradually becoming compact and overlying sound red shale. Some type of pile foundations were, of course, required. It has been our experience with rock as close to the surface as it was at Linden, and with heavy loads such as occur in a generating station building, that steel H piles provide the most economical foundations. There was, however, some question as to their durability at this location. Analysis of boring samples in the area of the building indicated that the organic silt and the material directly below it were high in acid content, in some locations with pH readings as low as 3.2. With this condition steel H piles, if used, would require protection of some nature. This would add to the cost and there would always be an element of uncertainty. It was, therefore, decided to use wood piles despite the fact that this type of pile required a considerably thicker mat and piers with wide bases under the heavier columns in order to distribute the load. An interesting fact in connection with this acid condition of the silt strata is that analysis of soil samples, taken from deep pits when the building excavation was under way, indicated that the acid condition had largely disappeared. The only explanation that could be given was that the acid condition was caused by chemical waste which had been dumped



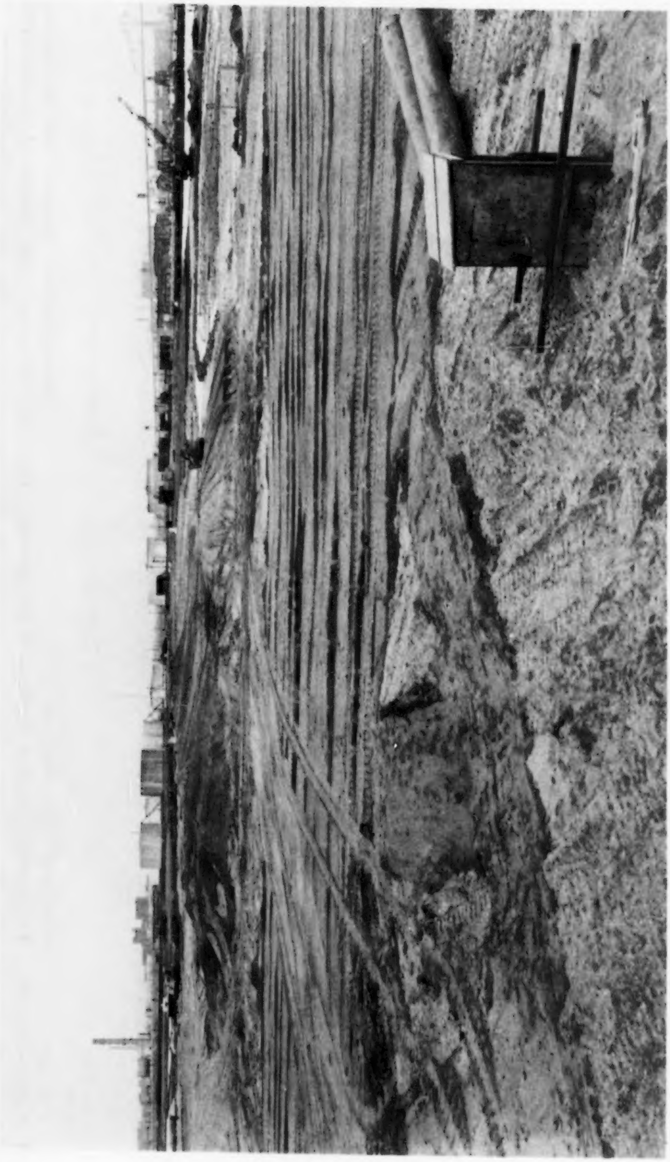
J.O. 8160 (18)
70595
10-25-54

Linden Generating Station Dredging Diversion to Piles Creek Public Service Electric and Gas Company

Fig. 5—Dredging diversion to Piles Creek.

J.O. 8160 (13) View of Plant Site - Looking East
70590
10-25-54 Linden Generating Station Public Service Electric and Gas Company

Fig. 6—View of the plant site. The suction dredge working at the handling basin can be seen in the center at the far side of the creek. At the right is the pipe line for depositing material on the site and also another suction dredge starting a new channel.



J.O. 8160 (36)
71050
12-21-54

Hydraulic Fill - East of Railroad - Looking North

Linden Generating Station

Public Service Electric and Gas Company

Fig. 7—Hydraulic fill east of railroad looking north.

J.O. 8160 (73)
71844
3-23-55 Linden Generating Station Break in Dike - Looking East Public Service Electric and Gas Company

Fig. 8—Break in dike looking east.

some years previously on the south section of the property and that as pumping to lower the ground water level was carried on, the acid was in part washed out of the silt. There was always, however, a bit of mystery about the condition. Despite the apparent clearing up of the condition, crushed limestone (Fig. 9) was placed over the building area before the mat was poured to serve as protection for the concrete.

The wood piles (Figs. 10, 11) used had a minimum tip diameter of seven inches and a minimum diameter three feet from butt of twelve inches. Steel shoes were used on the tips of the piles to insure adequate seating in the compact material overlying the shale strata. Pile load tests indicated that the piles could be loaded safely to eighteen tons and this accordingly was the design load used. A total of 8,800 piles were driven for the building foundations. In the Boiler House, and in other heavily loaded sections, they were spaced three feet on centers in both directions.

A continuous reinforced concrete mat was placed on top of the piles, varying in thickness from two feet six inches to five feet. It was designed to spread the loads over the required number of piles. Piers under the heavier loaded columns had bottom sections up to 200 square feet in area in order to reduce the bending moments in the mat and thus keep the amount of reinforcing within reasonable limits. In general, 1-1/4-inch square bars were used for the heavier sections of the mat.

The concrete design was based on using a material with a 28 day strength of 3000 pounds per square inch. The mix, however, was designed for a 28 day strength of 3500 pounds per square inch and frequent tests indicated that the material had an average strength considerably above this. The mix contained fly ash which has become standard procedure with our company on all major projects. The substitution of 100 pounds of fly ash for the equivalent quantity of cement per cubic yard of 3500 pound concrete has been found to produce a more workable and durable concrete and to give higher ultimate strength. On large jobs where the handling of a special material is not a large factor it is also more economical.

The construction of the mat was complicated by the 60 inch pipes (Fig. 12) for circulating water, which were located partially below and partially within the mat, and by the banks of electrical conduit running through the mat (Figs. 13, 14). These features were located as far as possible to clear the major bands of reinforcing steel. There were, of course, the pits under the condensers and the discharge tunnels, which were below the level of the main section of the mat and complicated the construction problems. A total of 1250 tons of reinforcing and 21,100 cubic yards of concrete were used for mat, foundation walls and piers.

The major portion of the concrete for the sub-structure was mixed on the site. To handle sand and aggregate a mooring dock was constructed near the mouth of the relocated Piles Creek. Here the materials were unloaded with a bucket crane (Fig. 15) and stock piled on areas adjacent to the dock. A batching plant was set up nearby from which point the materials were trucked to convenient locations around the outside of the foundation where mixing plants were set up. The mixers dumped into concrete buggies which were used for distributing the material. The mooring dock was constructed of creosoted piles and timber as a permanent structure to receive oil barges in the event it should be necessary.

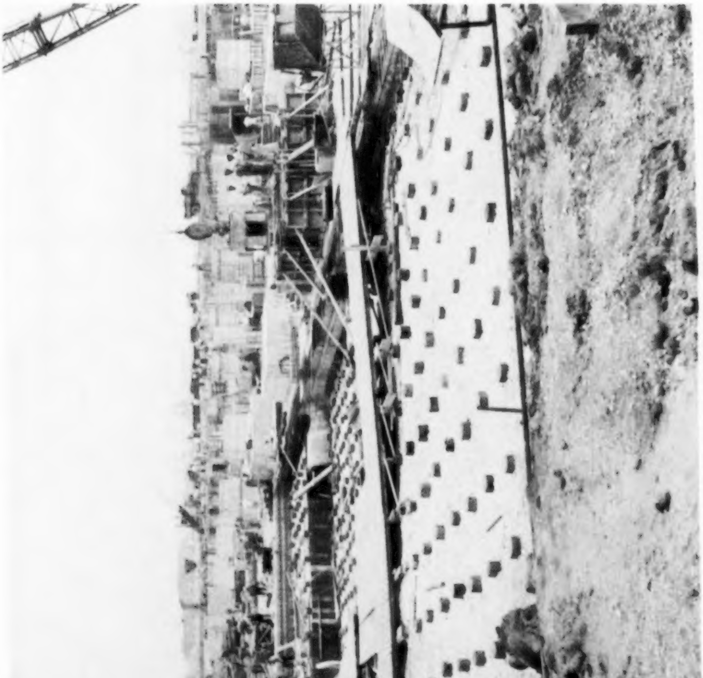
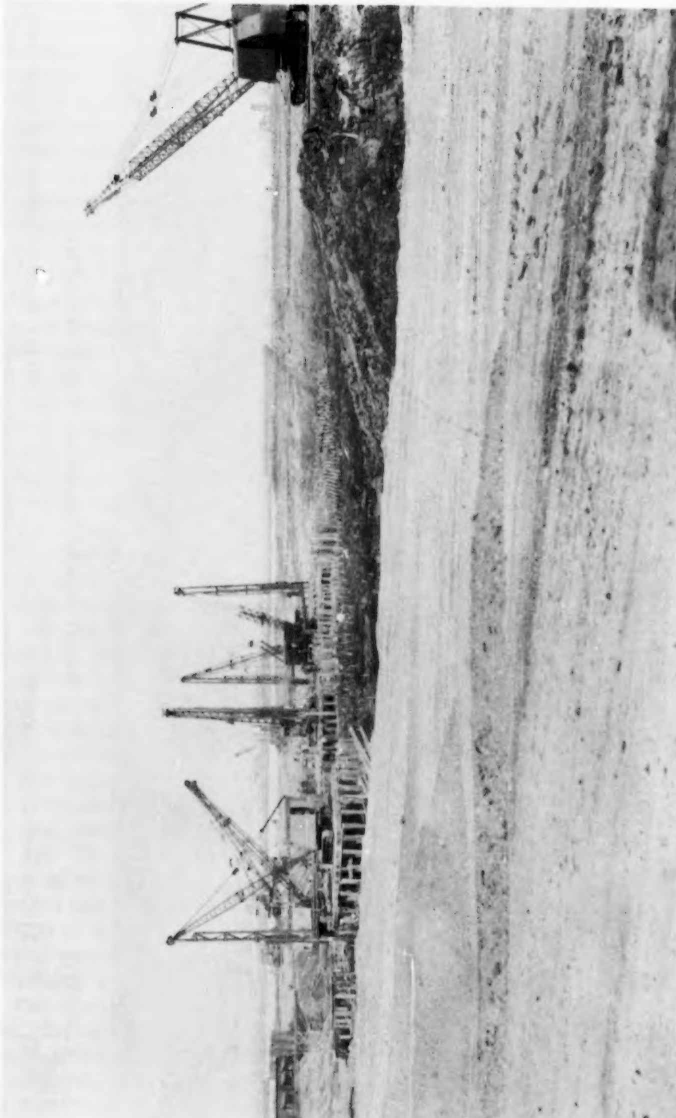


Fig. 9—General view of foundation work looking north.



J.O. 8160 (58)
71727
3-14-55 Linden Generating Station Driving Piles for Main Building Foundation Public Service Electric and Gas Company

Fig. 10—Driving piles for the main building foundations.

J.O. 8160 (101) Linden Generating Station General View Looking Northeast Public Service Electric and Gas Company
72410
5-20-55

Fig. 11—General view of main building construction looking northeast.

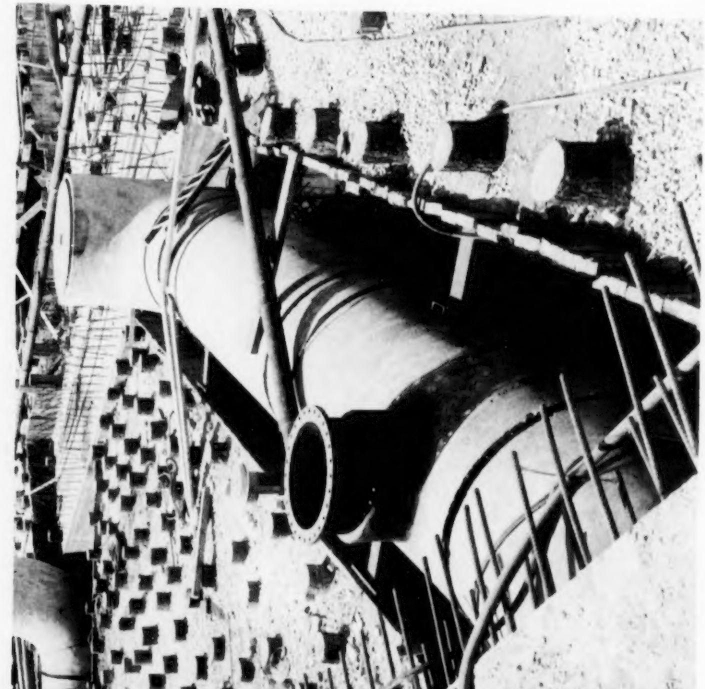
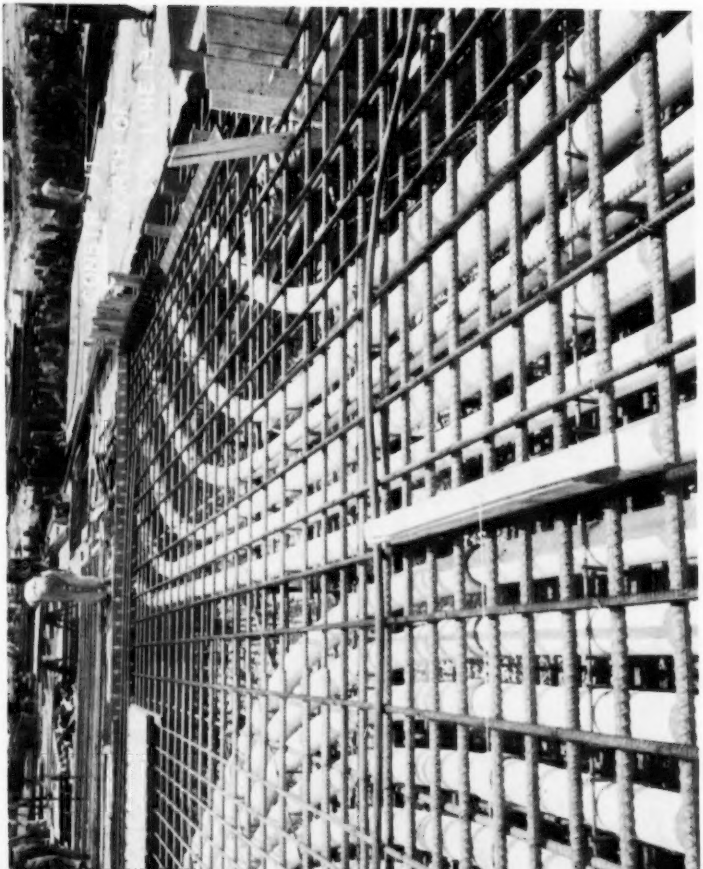


Fig. 12—Circulating water pipes.



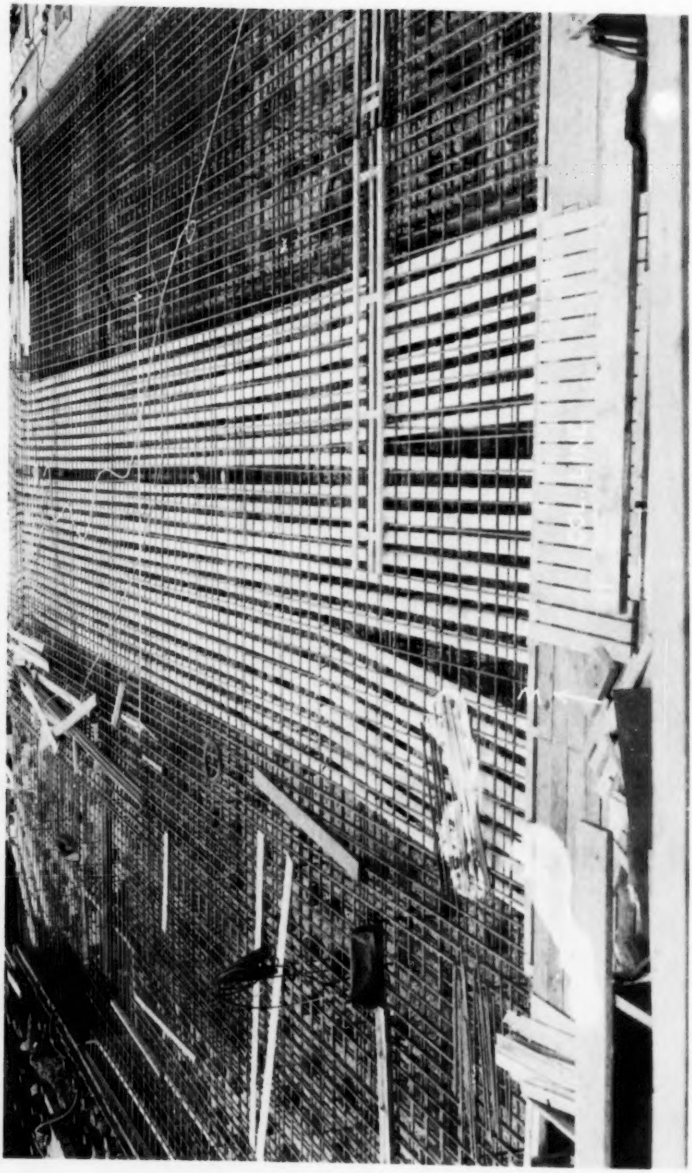
J.O. 8160 (91)
72400
5-16-55

Conduit In Mat - Elevation 88

Linden Generating Station

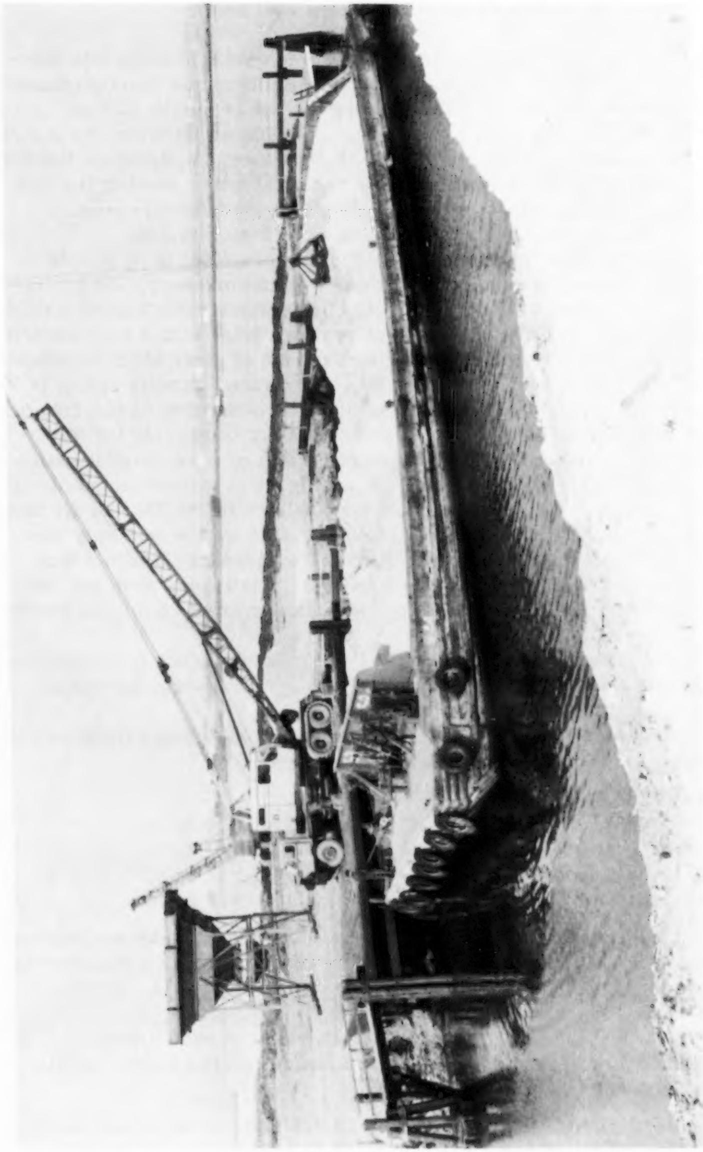
Public Service Electric and Gas Company

Fig. 13—Concrete mat and embedded electrical conduit.



J.O. 8160 (89) Conduit In Mat - Elevation 88 - From Cable Vault at Column 'B 11'
72398
5-10-55 Linden Generating Station Public Service Electric and Gas Company

Fig. 14—Concrete mat and embedded electrical conduit.



J.O. 8160 (60)
71729
3-14-55

Linden Generating Station Unloading Sand for Concrete Batching Public Service Electric and Gas Company

Fig. 15—Unloading sand for concrete batching.

The Main Generating Station Building

The principal structure on the site is the power station building which includes the Boiler House, Turbine Room, Service Building and Demineralizer Room. The overall dimensions of the building for the two units, are approximately 274 feet by 390 feet, with a twelve foot deep basement under the entire structure. The elevation of the grade floor is designated as elevation 100 feet which is determined by the highest previous recorded water level at the site, plus 18 inches. Other buildings on the property include Water Treatment Building, Electrical Control House, Gate House and Meter House.

The architectural treatment (Fig. 16) oi the main building is of simple contemporary design, as are the other buildings on the property. As far as consistent with power house architecture, the projections were kept to a minimum. In general, the exterior walls are of red face brick with limestone trim, except the Turbine Room which has a horizontal band of glass block between the crane rail and ceiling on both side walls. Corrugated transite siding is used to enclose the trusses above the glass block on both sides of the Turbine Room, and is returned on the permanent end. In order to provide for future extension, the entire exterior of the temporary end is of corrugated transite. Aluminum windows are used throughout. A panel type construction was used on the permanent end of the Turbine Room and the side of the Service Building. On the end of the Turbine Room an area of 40 feet high by 100 feet long was finished with panel type construction of alternate squares of glass and blue colored aluminum panels. On the Service Building, continuous sash are used on floors at elevation 120 feet and 136 feet with blue colored aluminum panels between.

Building maintenance costs are reduced by the extensive use of glazed tile for interior finish and also by ventilation under slight pressure, for cleanliness.

The structural frame work for all buildings is of steel without fireproofing and was designed on the basis of the following stresses:

Boiler House and Turbine Room Roof Trusses	- 18,000 lbs per sq in.
Stack supporting steel	- 16,000 lbs per sq in.
Induced draft fan supporting steel	- 12,000 lbs per sq in.
All other steel	- 20,000 lbs per sq in.

Owing to space limitations the five self supporting steel stacks are carried on steel framing over the air heaters (Fig. 17). Each stack has a diameter of 12-ft 5-in. inside of steel shell and a top elevation of 225-ft 0-in. Length of each steel stack is 157-ft 1-1/4-in. Stacks are lined with 2-1/2 inch thick gunite, made with Lumnite Cement and trap rock sand. A wind pressure of 25 pounds per square foot of projected area was assumed on the stacks and the following stresses were used in the design:

Tension on steel plates	- 12,000 Lbs per sq in. net section.
Compression on steel plates	- 9,000 Lbs per sq in. gross section.
Rivet shear	- 9,000 Lbs per sq in.
Rivet bearing	- 18,000 Lbs per sq in.
Anchor bolts	- 15,000 Lbs per sq in.

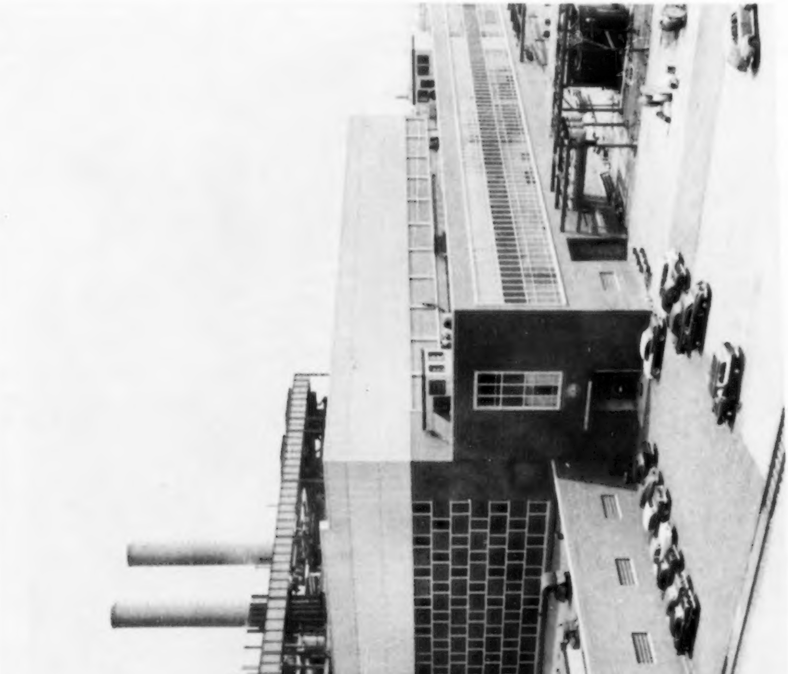
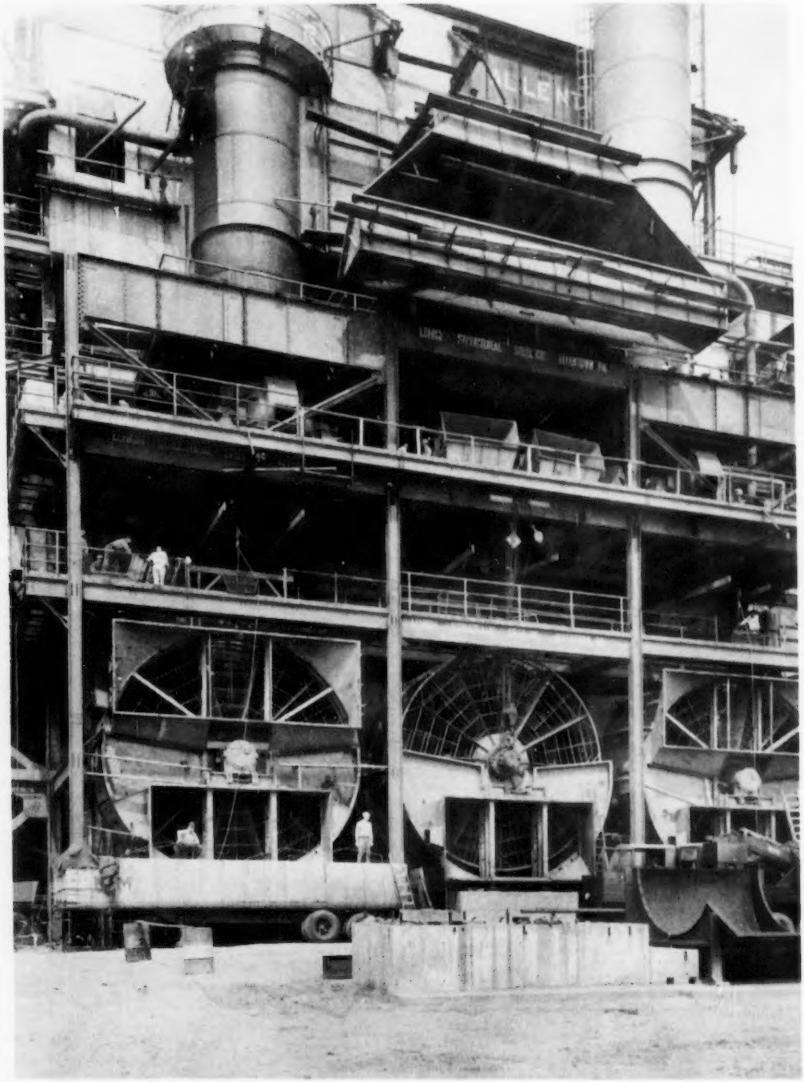


Fig. 16—General view of main building.



J.O. 8160 (1231) Southeast Corner Boiler House
81128
6-6-57 Linden Generating Station Public Service Electric and Gas Company

Fig. 17—Southeast corner of Boiler House showing FD fan and stack supports.

The base of each stack is anchored to supporting girders by sixteen bolts three inches in diameter. The wind shear on the stack is carried down to the foundation through a system of wind bracing between boiler house columns in one direction and bracing in stack supporting steel in the other direction.

The general layout of the Boiler House and Turbine Room is shown in Fig. 18.

Because of the great demand for structural steel and the extended lead time required by the mills, it was necessary to start steel design as soon as the boiler sizes and loadings were obtained. In order to do this, the weights and distribution of loads for auxiliary equipment and piping had to be estimated and for this reason the steel was designed for 18,000 pounds per square inch instead of the allowable 20,000 pounds per square inch. In this manner the steel was obtained on time. When complete information was available, affected beams were rechecked with allowable stresses of 20,000 pounds per square inch. This not only saved time in getting the structural steel, but proved more economical due to saving field changes to strengthen the steel. Further economies were obtained by using high strength bolts for all field connections of structural steel.

The boiler tubes, steam drums and boiler casings are hung from the girders above, the top of which are at elevation 224-ft 9-in. (Fig. 19). These boilers are carried to the foundation by six columns per boiler, one at each corner and one intermediate column on each side. The maximum loads occurred at the middle columns. The largest load at the top of the column including the supporting steel, is 1277 kips for the first Unit and 1324 kips for the second Unit.

The stability of the Boiler House and Turbine Room is provided for by vertical cross bracing between the boiler columns. Since the Boiler and Turbine columns do not line up, the transmission of horizontal forces from the Turbine Room and Service Building was accomplished by providing a horizontal truss between the boiler columns and turbine room columns at the approximate location of the bottom chord of the Turbine Room roof trusses.

Turbine Room

The size of the Turbine Room was governed by the space required for the installation of the turbine generators and auxiliary mechanical (Figs. 20, 21, 22 and 23) and electrical equipment, and is 128-ft 0-in. wide by 277-ft 0-in. long, outside dimensions, and is divided lengthwise into six equal column bays 45-ft 0-in. wide.

The turbo-generators are at elevation 120-ft 0-in. A 200-ton capacity crane spanning 120-ft 0-in. center to center of crane rails is supported on girders with top of crane rail at elevation 154-ft 0-in. An arched ceiling made of perforated mineral tile is suspended from the lower chord of the trusses.

The space between the trusses was utilized to provide ventilating equipment for Boiler House and Turbine Room. Six ventilating rooms, complete with filters, washers, fans and motors were installed in this area affecting a considerable saving in building cost. Further economy was derived from being able to use corrugated transite for the walls of the Turbine Room above the suspended ceiling. Another saving was made possible by designing the turbine columns and roof trusses as "L" frame structures from the base of the Turbine Room columns on the Service Building side to the turbine columns on

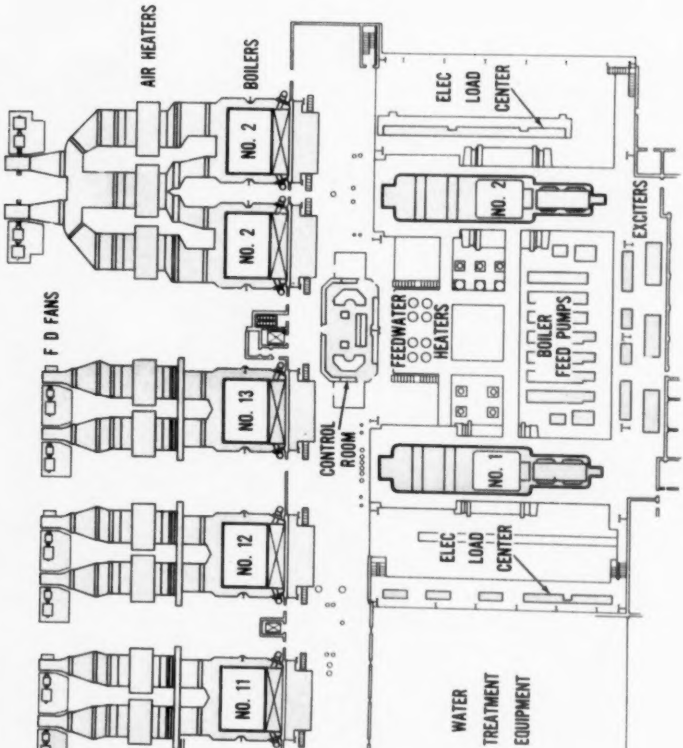


Fig. 18—Boiler House and Turbine Room layout.

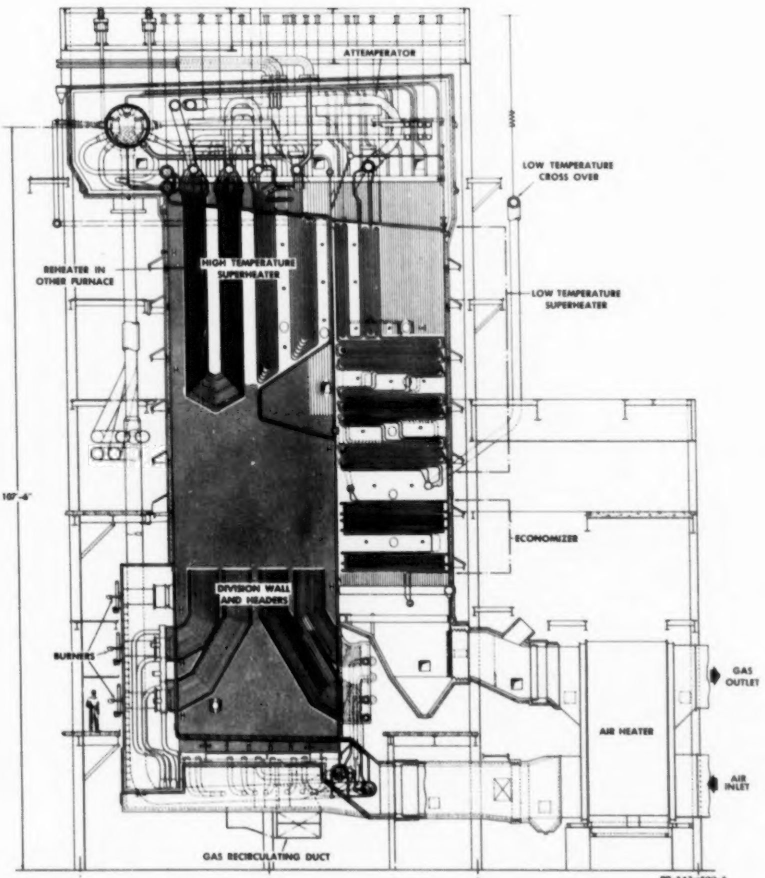
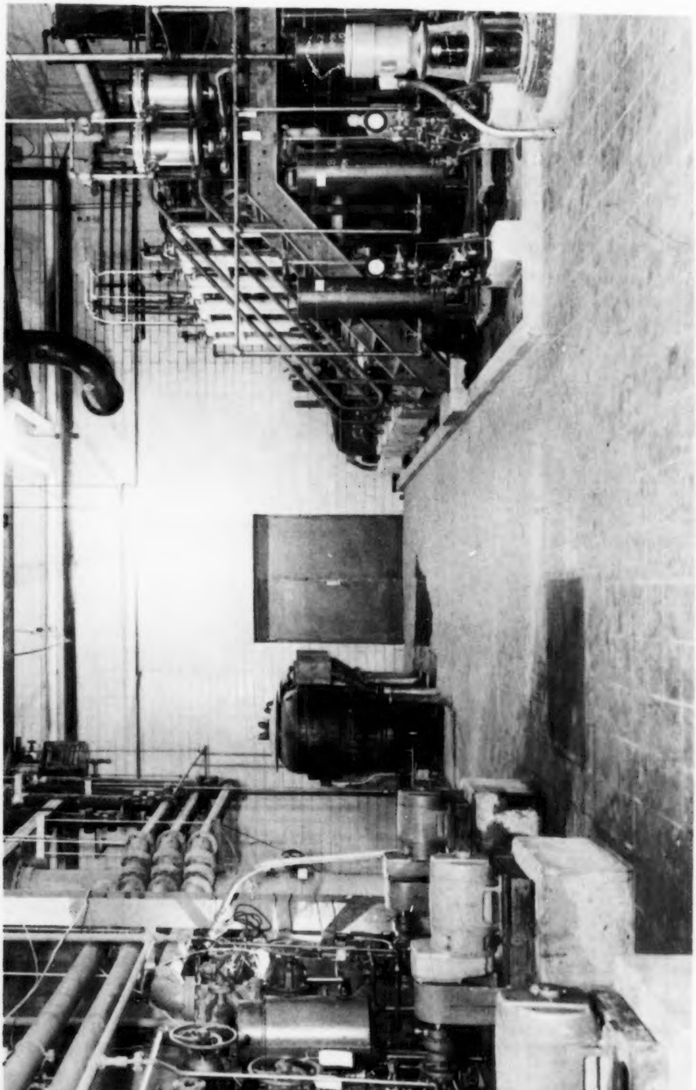
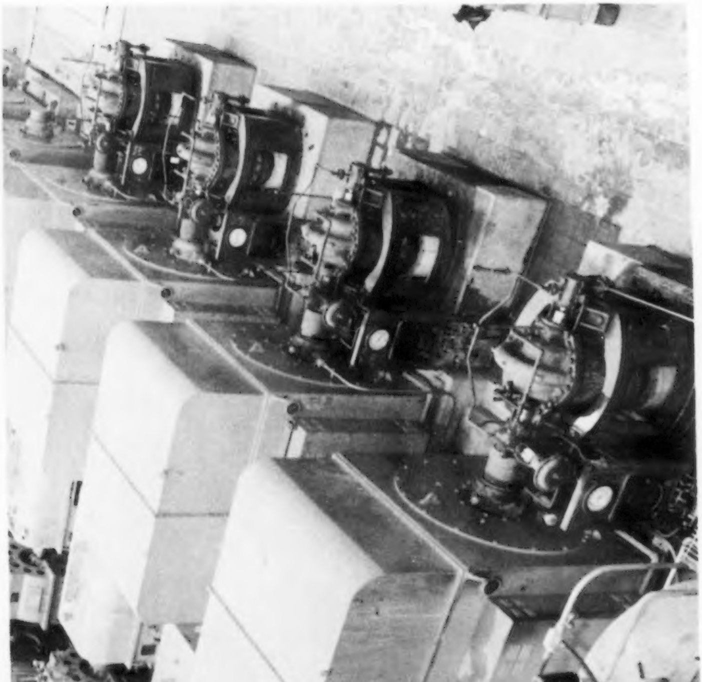


Fig. 19—Boiler for Unit No. 2



J.O. 8160 (1250) Chemical Feed Station and Vacuum Pumps
81530
7-5-57 Linden Generating Station Public Service Electric and Gas Company

Fig. 20—Chemical feed station and vacuum pumps.



Boiler Feed Pumps

Public Service Electric and Gas Company

J.O. 8160 (1218)
80635
5-6-57

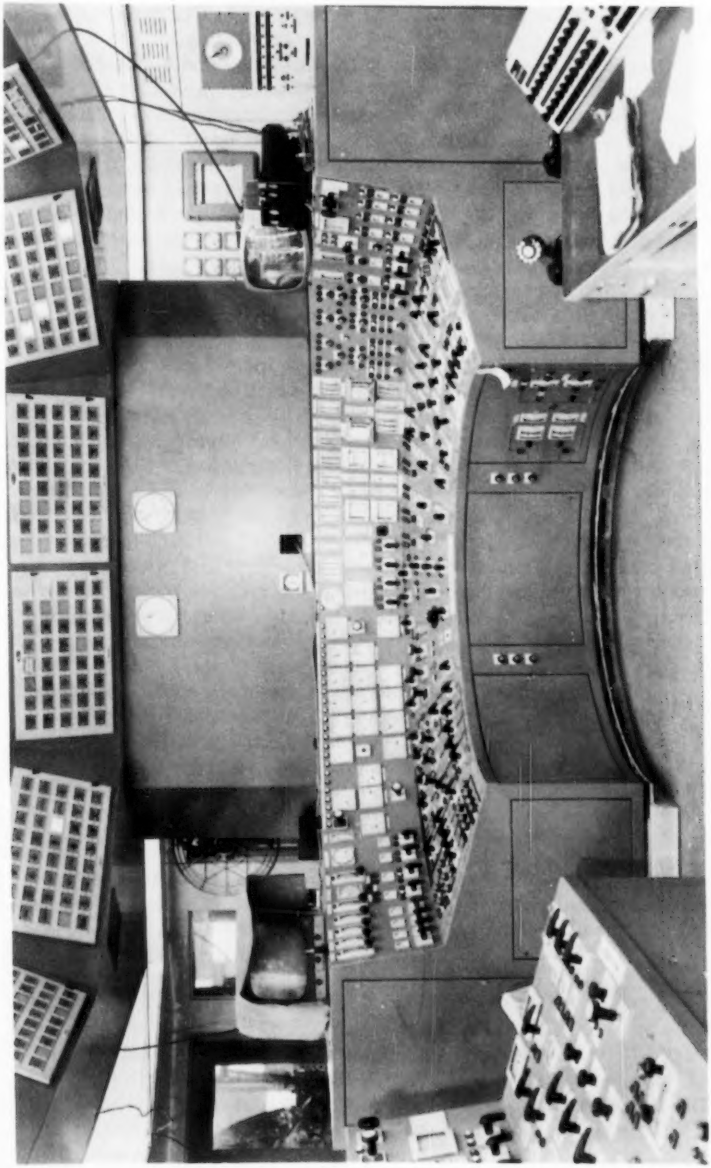
Linden Generating Station

Fig. 21—Boiler feed pumps.

J.O. 8160 (1239)
81341
7-5-57

Linden Generating Station Exciter Bay Public Service Electric and Gas Company

Fig. 22—Exciter bay.



J.O. 8160 (1258) No. 1 Control Panel Public Service Electric and Gas Company

81913
8-6-57 Linden Generating Station

Fig. 23—Unit No. 1 control panel in control room.

the Boiler House side. The vertical forces were taken by the opposite turbine column and the horizontal forces taken by the boiler bracing.

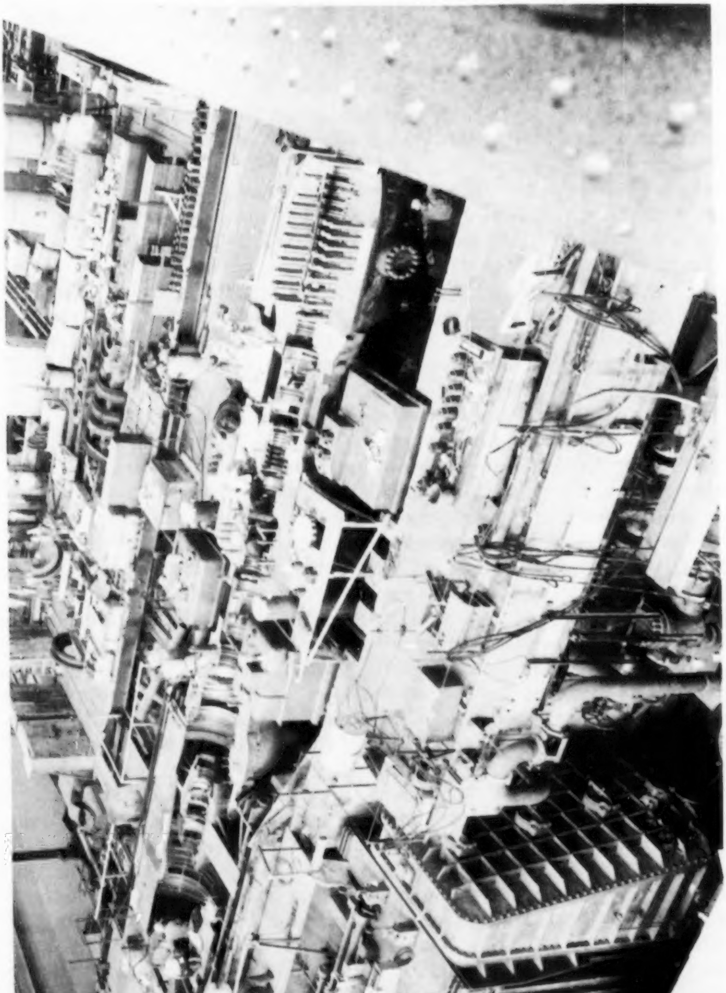
The intermediate trusses were designed as simple beams supported at each end by longitudinal trusses between the Turbine Room columns. The roof trusses were stiffened laterally by three rows of trusses running the length of the building.

For the erection and operation of the Turbine Room equipment one two-hundred ton capacity crane was installed. The runway girders supporting the crane were designed for maximum vertical wheel load plus twenty-five per cent impact and a horizontal impact of ten per cent of live load plus weight of trolley acting along the longitudinal axis of the crane. The impact in the direction of crane travel was assumed as ten per cent of the maximum total reaction of the crane, and is taken by the longitudinal bracing in the sidewalls of the Turbine Room.

The turbo-generators (Fig. 24) are supported on individual concrete foundations at elevation 120-ft 0-in. These foundations are entirely isolated from the main building steel above the basement floor at elevation 88-ft 0-in. The design of the turbo-generator foundation was based principally on deflections, care being taken that the deflection along the longitudinal axis of the foundation was uniform under each bearing. The allowable variation in deflection being .02 inch. The loadings upon which the design was based consisted of all vertical loads on the foundation plus one hundred per cent of machine loads for impact. Longitudinal force, furnished by manufacturer, applied at center line of shaft (approximately 50 per cent of the weight of the machine) and transverse force at each bent, equal to 50 per cent of the machine weight supported by the bent and applied at machine center line. To prevent vibration the foundation was designed with heavy column sections, deep girders and concrete walls where permissable, resulting in a weight of the concrete foundation of approximately twice the weight of the machine. The requirement for the design of the foundation resulted in very low stresses, however, the reinforcing used amounted to approximately 200 pounds per cubic yard of concrete.

Miscellaneous Structures

There are, of course, numerous other structures in a power plant besides the main generating station building. There are, for instance, those required for handling the condensing water, including the intake canal, the screen well and discharge tunnel and canal. At Linden the intake canal is located along the north property line and is 30 feet wide with the bottom 30 feet below grade. It has steel sheet piling both sides with steel braces tying the sides together. Lateral stability is obtained by the use of steel H batter piles welded to gusset plates connected to the steel walers. Steel batter piles make a neat and effective way of taking care of the horizontal loads in a bulkhead type of structure. The limiting and somewhat uncertain element in the design of a structure of this type is the ability of the steel H piles or steel sheeting to take uplift. Our experience in driving steel piles into shale strata is that under heavy driving the steel beams mesh into the rock and develop a good grip. To insure permanence, both steel H piles and batter piles were thoroughly cleaned before driving and given two coats of bituminous paint applied in accordance with a rigid specification. The structure also has cathodic protection.



J.O. 8160 (1263)
82113
8-21-57

Linden Generating Station

Turbine Room - Looking North

Public Service Electric and Gas Company

Fig. 24—Turbine Room looking north showing turbine generators being erected.

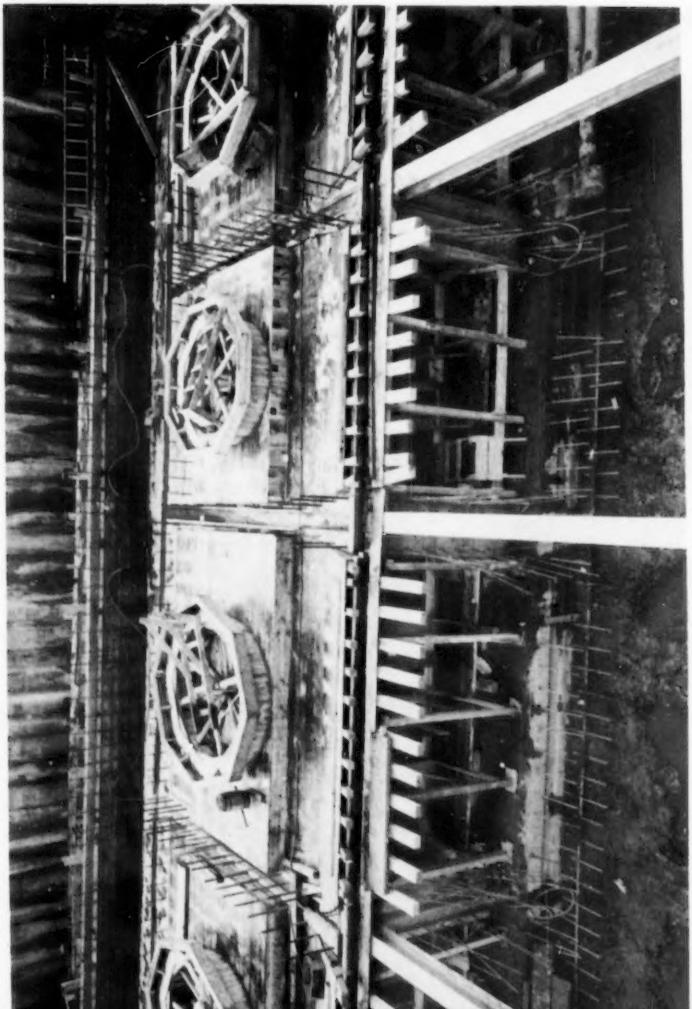
The screen well (Fig. 25) is a concrete structure 41 by 65 feet, containing trash racks, gates, screens and circulating pumps. Its bottom mat is supported by steel H piles and is 30 feet below grade. Steel H piles could be used with safety here as they were cut off below the contaminated soil strata. The problems involved in its construction were those encountered in any complicated concrete structure built within a cofferdam. It contains about 1500 cubic yards of concrete.

The discharge tunnel and canal carry the condensing water from the building across the yard to relocated Piles Creek. The tunnel is used for the section adjacent to the present building and continued to a point beyond the end of the building extension required for installation of the next unit. From this point on, the discharge is carried in an open ditch, the sides of which are faced with rip-rap. The top of the discharge tunnel is located, in general, 12 feet below grade with its bottom 22-ft below. It is a concrete box structure (Fig. 26) with intermediate center wall having an overall width of 20 feet. In constructing it, the excavation was carried in open trench to the level of the top of the tunnel, at which point wood sheet piling was driven on the lines of the outside faces of the tunnel where it was also used for forms for the concrete walls. It proved to be an economical construction.

In addition to the major structure, foundations were provided for many minor ones and for much of the equipment; the air heaters and forced draft fans back of the boilers, the water storage and treatment tanks just north of the main building, electrical equipment both adjacent to the main building and in the switch yard, and oil storage and metering tanks in the northwest corner of the property. All of these, of course, required wood pile foundations. The kind of wood pile was determined by economical considerations. In general, if the layout of the foundation fixed the bottom of the concrete pier or mat less than four feet above mean tide in the adjacent Arthur Kill, the concrete would be extended lower than required, and plain wood piles used with the cut-off one foot above mean tide. Where the required location of the bottom of the concrete pier or mat was higher than four feet above mean tide, creosoted wood piles were used with the cut-off fixed by the design of the concrete. These creosoted piles were purchased under a rigid specification and with inspection at the creosoting plant. The water and oil tank foundations consisted of concrete mats 1-ft 6-in. thick, with a two inch sand layer between concrete and tank bottom. The oil tanks were surrounded by concrete dikes made up of rectangular precast sections and wrapped with prestressed wire cables covered with gunite (Figs. 27, 28). Altogether there were approximately 15,000 cubic yards of concrete poured for the equipment foundations.

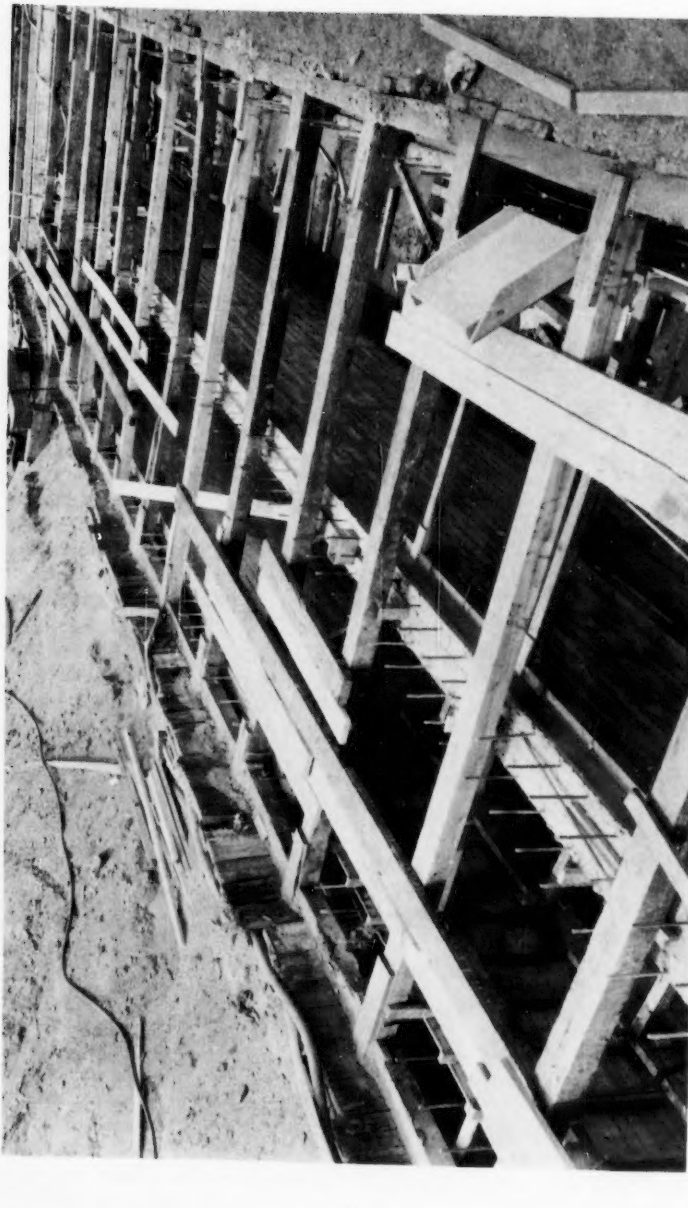
As the yard was and is still settling due to the placing of the fill, it was necessary to provide supports for all rigid yard piping and conduit, except under the railroad track where the fill had been in place for many years. This, of course, made a considerable amount of detail design work and necessitated tearing up large sections of the yard in the course of construction. Wood piles, either plain or creosoted, were used for all of these supports, with the kind being determined by the location in relation to ground level of the pipe or conduit to be supported. Underground conduits were supported on continuous mats (Fig. 29). The supports for the steam lines running to the Esso Refinery, together with the tracers at oil lines, were designed for the thrusts and moments due to temperature changes.

One problem which confronted us at Linden was the corrosive nature of the atmosphere due to the presence of many chemical plants in the neighborhood.



J.O. 8160 (293) Linden Generating Station View of Screen Well - Looking South Public Service Electric and Gas Company
74553
11-21-55

Fig. 25—View of screen well looking south.



J.O. 8160 (159)   View Looking North Along Discharge Canal
73252
7-20-55   Linden Generating Station   Public Service Electric and Gas Company

Fig. 26—View looking north along discharge canal.

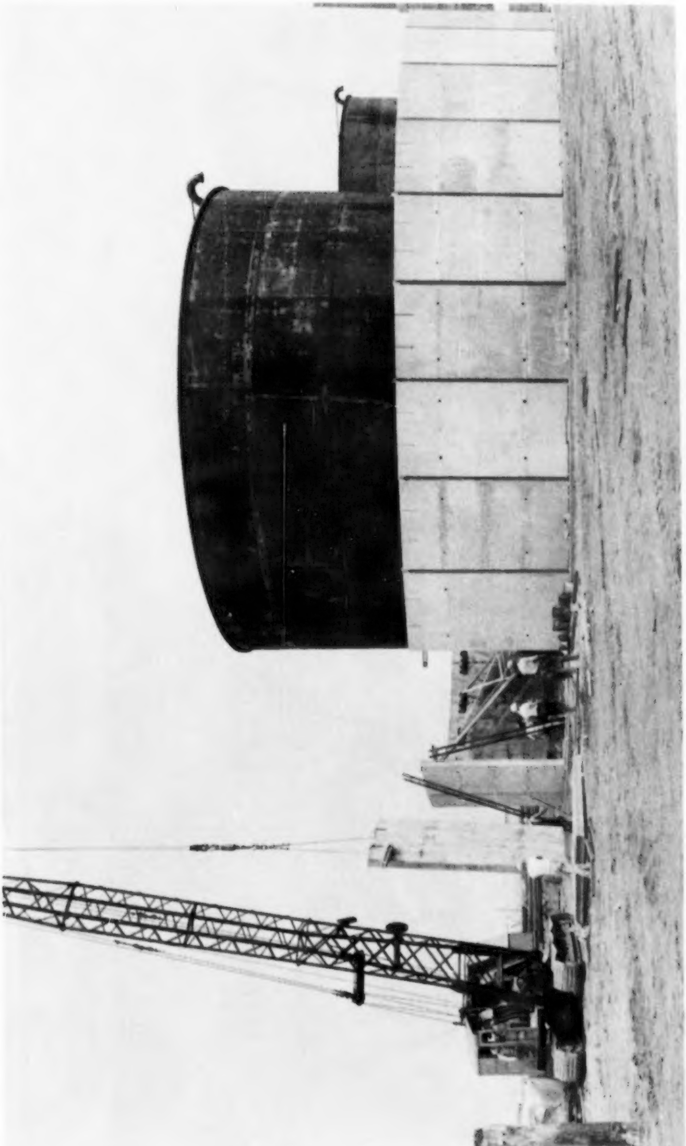


Fig. 27—Concrete dike showing precast sections.

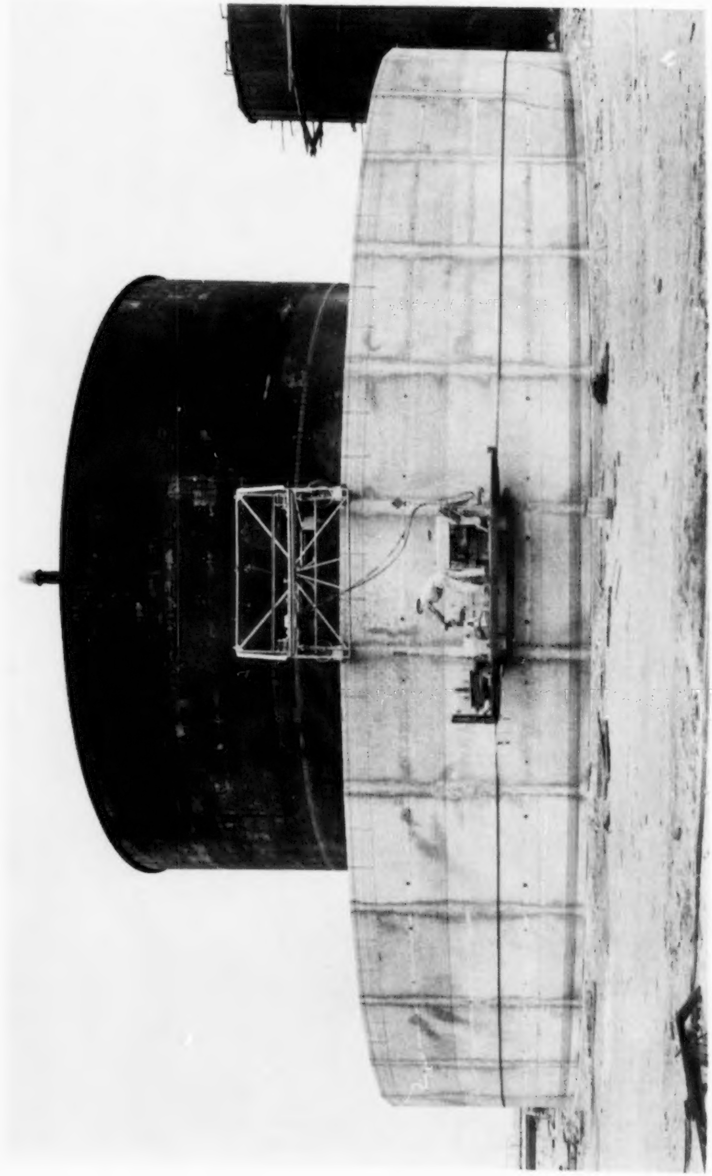


Fig. 28—Prestressing concrete dike.

J.O. 8160 (337)
74738
12-14-55

Linden Generating Station Duct Runs Transformer Yard

Public Service Electric and Gas Company

Fig. 29—Duct runs showing continuous mat.

Experience with galvanized steel towers in the area had indicated that galvanizing provided only temporary protection for steel work. It was then decided to use painted steel for outdoor structures and supports. The specifications for this painting were as follows:

The fabricated steel was acid pickled to remove all mill scale and rust. The members were then washed in hot water, dried and wire brushed to remove any surface film. After this treatment a chemical resistant type shop primer was applied by airless hot spray equipment to all surfaces in a one-pass coat. A minimum thickness of 2.0 mils (.002 inches) was specified.

After erection, the steel was hand cleaned and spot primed where necessary with the same type priming paint. Nuts, bolts and steel connections were pretreated with a rust preventive solution prior to priming. Finally, the structure was given two brush coats of chemical resistant finish paint, the first coat somewhat lighter in shade to insure complete coverage. It is expected that repainting of the steel will not be necessary for a period of ten years, or more.

CONCLUSION

The role of the Civil Engineer in the design of a power station is in one sense a minor one for his function is merely to provide the structures necessary for electrical and mechanical equipment. They are a means to an end rather than an end in themselves. But the fact that the work of the Civil Engineer must be done within the limitations imposed by the electrical and mechanical requirements calls for a resourcefulness not always required in other branches of his work. Each new project seems to bring new problems both in design and construction which kindle interest and challenge his ingenuity.

Paper 1683 June, 1958 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: PERFORMANCE AND MAINTENANCE OF DIX RIVER DAM[a]

Lewis A. Schmidt, Jr.,[1] M, ASCE
(Proc. Paper 1683)

FOREWORD

This paper is one of a group from the Symposium on Rockfill Dams, June 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element and is described approximately as follows: a dam in which at least 50% of the maximum section is quarried rock; and in which at least half of the rock is dumped from lifts rather than placed in layers. This includes the types with impervious face membranes, sloping earth cores, thin central cores and with thick cores as limited roughly by the above description.

The objective of the Symposium is to assemble up-to-date information on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Constructed to a height of 275 feet in 1923-25 Dix Dam was then the highest rockfill dam built. Over a period of 33 years considerable data on leakage, maintenance, condition and settlement have been accumulated. This information is presented herein for applicable consideration in the design of similar future projects.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1683 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 3, June, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June, 1958.

1. Pres., Schmidt Eng. Co., Inc., Chattanooga, Tenn.

History

Dix Dam is a rock fill dam, which was built in 1923-1925 on the Dix River in Central Kentucky approximately 3 miles upstream from its confluence with the Kentucky River. It was built for the Kentucky Hydro-Electric Company, which later became the Kentucky Utilities Company, the present owner. The project was designed by Mr. L. F. Harza, the founder of the present Harza Engineering Company Inc. of Chicago. Consultants were Arthur P. Davis and M. M. O'Shaughnessy. A location map is shown in Figure 1.

Design Features

The crest of the dam was built to elevation 780 with 5 feet of superelevation in the deepest section for settlement allowance. Normal river bed level was from elevation 505 to 515. The dam is 275 feet high in its highest portion, 1020 feet in length along the crest, with the upstream slope built on a vertical curve starting at 1 on 1.2 at the base and ending with 1 on 1 at the crest. The downstream slope is uniformly 1 on 1.4. The crest is 20 feet wide and follows a compound curve of 1700 and 10250 feet radii at the east and west sides respectively. The final volume of the rock fill was 1,756,000 yards of which 70,700 cubic yards were accounted for by the derrick laid rock section. A plan of the dam and spillway is shown in Figure 2 and details of the dam through the maximum section are shown in Figure 3.

A spillway channel, 250 feet wide, was designed to be cut into the west or left bank, flaring out to a crest length of 800 feet at the actual spillway lip. Rock to construct the dam was obtained from this spillway cut which is 2000 feet long.

A 24 foot diameter, concrete lined, horseshoe shaped tunnel, 875 feet long, shown in Figure 4, was driven in the east cliff for the initial purpose of diversion during construction and for the ultimate purpose of carrying the water from the reservoir to the powerhouse. At the upstream end of the tunnel a chimney type intake tower was constructed and 125 feet upstream from the downstream portal of the tunnel a concrete plug, 35 feet long, was built to contain the entrance to three 8 foot diameter penstocks which connect to a 30,000 horsepower powerhouse of three equally sized hydroelectric generating units. A derrick laid rock section, 14 feet thick at the base, and 4.5 feet thick at the crest, was constructed as the base for a reinforced concrete diaphragm of 18-inch thickness at the cutoff trench and 8-inch thickness at the crest. A cutoff trench 8 feet wide and of depth to good sound foundation rock was excavated in rock and concreted back. Up to elevation 670 a 3-inch T and G timber face was left in place outside of the concrete diaphragm.

The original spillway consisted of a modified concrete rollway at elevation 750. Soon after operations began flashboards were installed on this spillway. In 1930 it was decided to raise the maximum operating level of the reservoir from 750 to 760. At this time 10 gates, 10 feet high and 35 feet clear opening were installed over a distance of 394 feet of the spillway. These gates were operated from above by two gantry cranes. The balance of the spillway, or 277 feet was raised by proposed collapsible concrete sections, which were designed to go out only in case of an unusual flood condition. By 1949 it was felt that the collapsible section should be replaced and an Ogee section at elevation 758.5 feet with 1 1/2 foot of flashboards was installed at that time.

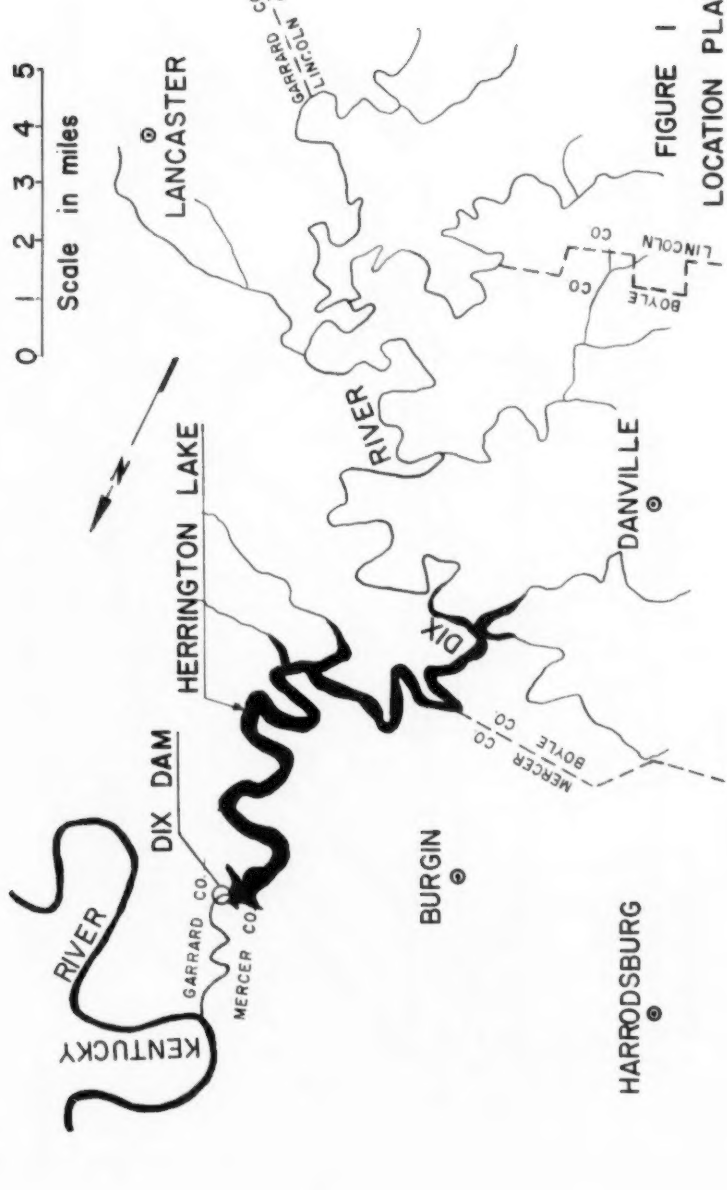


FIGURE 1 LOCATION PLAN

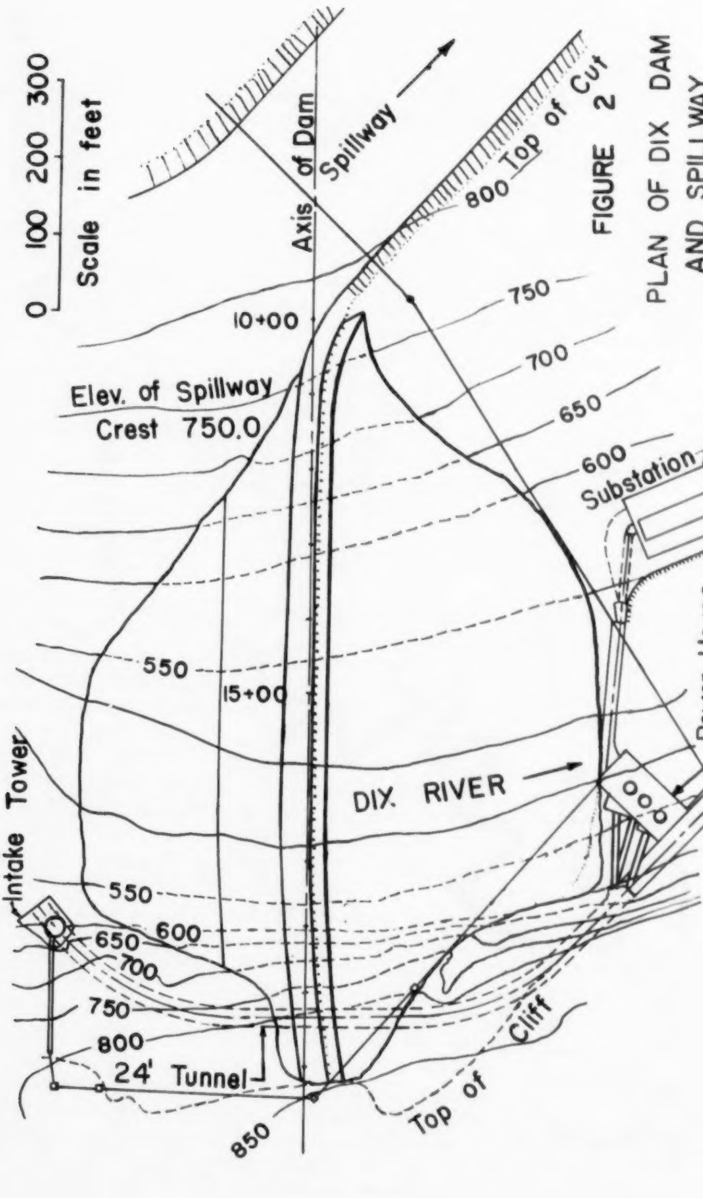


FIGURE 2
PLAN OF DIX DAM AND SPILLWAY

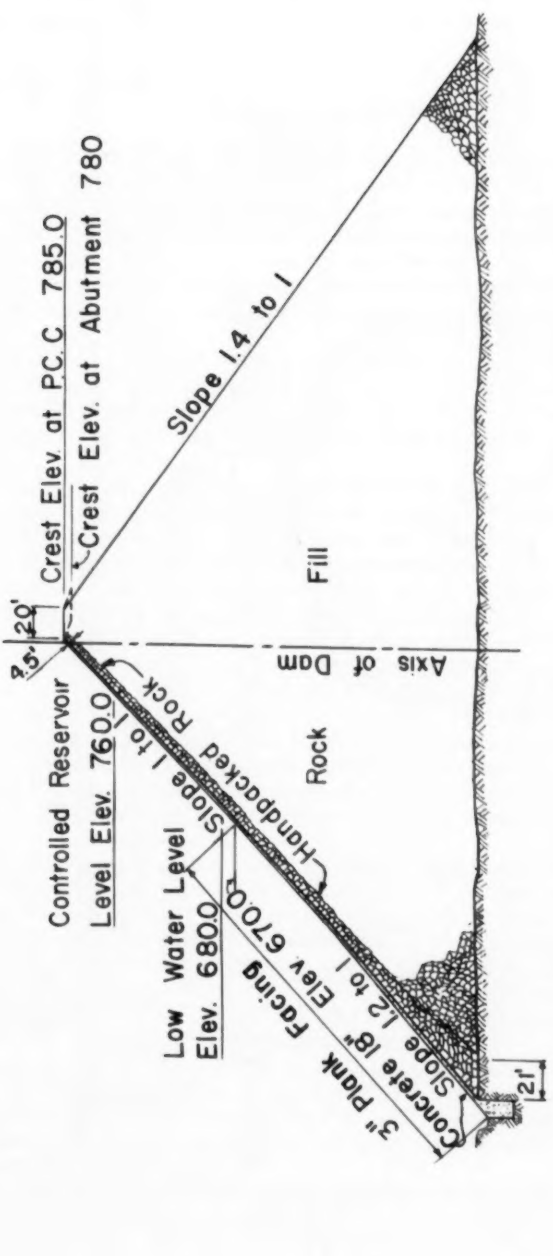


FIGURE 3
MAXIMUM SECTION

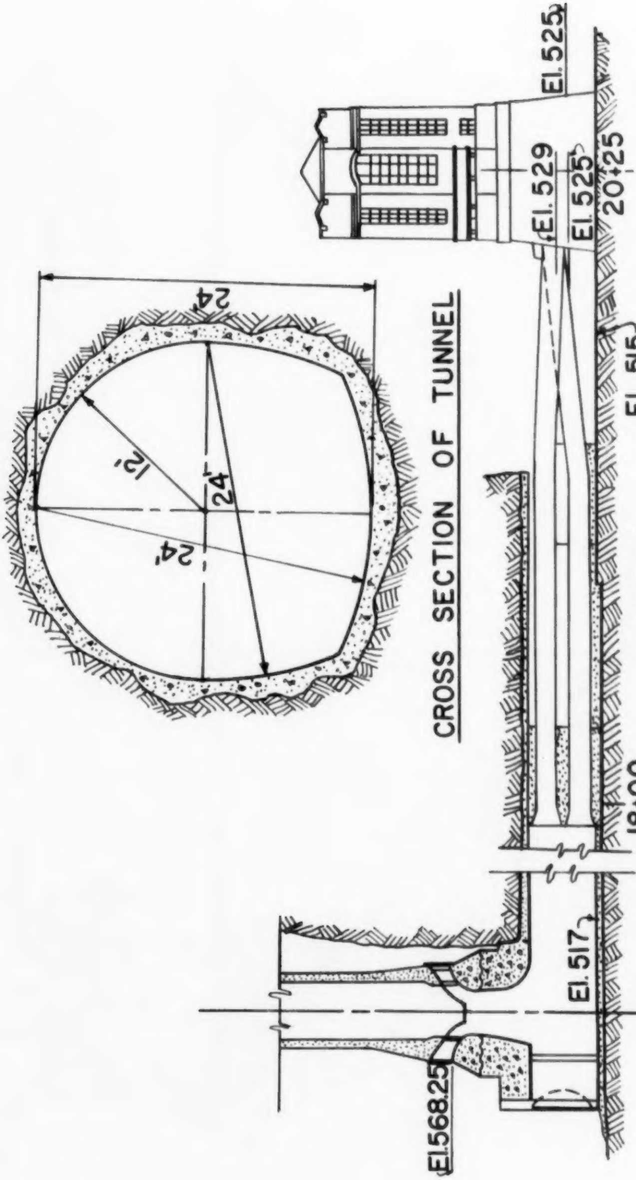


FIGURE 4

DETAILS OF TUNNEL

A section through this portion of the spillway is shown in Figure 5. Additional spillway capacity at the gates was provided by lowering the gate seat levels. The gates were made larger and reinforced to meet this additional head of water. Two gate seats were lowered 1 foot each, two were lowered 2 feet each and three were lowered 1 1/2 feet each with three remaining at the original level.

Construction Features

All talus material was stripped from the east bank and wasted. Similarly all the west side flood plain was stripped and the material wasted. This work was done with steam shovels and narrow gage railroad equipment. At the start of the job it also was planned to strip the entire spillway before loading out rock for the fill. In the interest of speeding up the work, portions of the stripping were omitted and this material was loaded out with the top cut of rock. Material in cars containing a majority of earth was wasted - other cars were dumped in the fill.

Rock excavation in the spillway channel was done by conventional drilling and blasting and rock loading was done with 3 1/2 cubic yard steam shovels of the railroad type which were converted to caterpillar mounting on outriggers at the front and a single rudder type, double caterpillar mounting at the rear.

Transportation was by 16 yard standard gage side dump cars, hauled by saddleback steam locomotives on a system of switchback railroad tracks over which trains traveled from the top of the west bank to the valley below.

Upon completion of overburden stripping from the base of the proposed dam, rock was side dumped from the west, or left bank switchback track at the 634 level, which was approximately 120 feet above the level of the river bottom. Figure 6 shows this operation. Side dumping and throwing of track was continued outward at this level until the fill reached almost across the canyon. At this time an 80,000 pound black powder coyote shot was pulled in the east cliff, just downstream from the downstream outline of the dam, which accounted for about 50,000 cubic yards of rock within the design limits of the dam. Later a well drill, dynamite shot of similar proportions was pulled in the east bank just upstream from the upstream outline of the dam and resulted in approximately the same volume of rock within the confines of the dam.

With this level at elevation 634 effected by side dumping, and the river diverted through the tunnel, a second lift of rock fill was then started. This lift was built by constructing a trestle above the 634 level to elevation 704 across the valley along the axis of the dam. From the level of this trestle rock cars were dumped for the full length of the trestle, the material being allowed to drop until the natural angle of repose had reached to the trestle level after which track was shifted both upstream and downstream until the fill met the required design outlines of the dam.

When the 704 level lift had been completed, a similar trestle was erected above this to elevation 774 and the rock fill was dumped from this level the same as from the lower level. The 774 level trestle is shown in Figure 7. The final crest elevations of 780 at the ends and 785 at the highest point were obtained by jacking track.

Throughout the filling operations, sluicing monitors were used to sluice

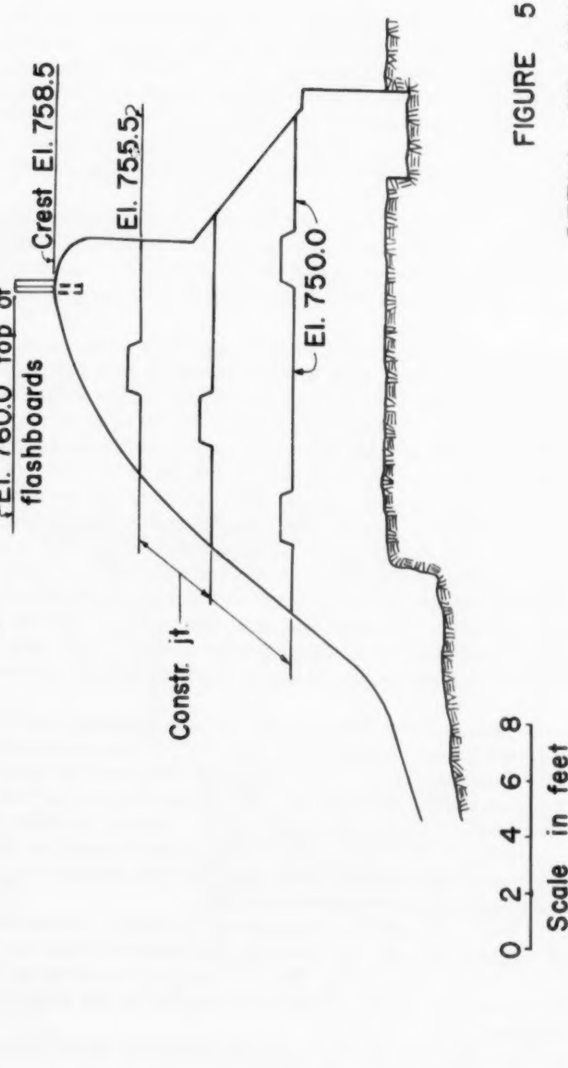


FIGURE 5

DETAIL OF SPILLWAY

FIGURE 6
Side dumping from the 634 level on the west side

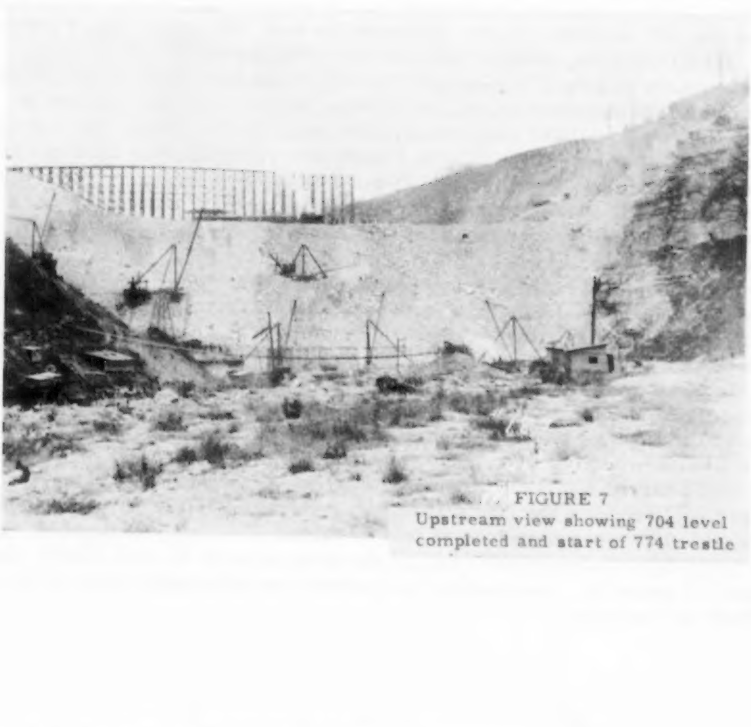


FIGURE 7
Upstream view showing 704 level completed and start of 774 trestle

down the fill. However candor must concede that this operation by comparison with the volume of fill placed was probably not as effective as it was intended to be.

All rock was hauled from the spillway area except that produced by the two east side shots which accounted for hardly more than 100,000 cubic yards and a small amount from an east side quarry which was soon abandoned as an uneconomical operation. Except to hold a 250 foot channel width and an 800 foot lip length, the spillway grades were varied to suit the amount of rock required to complete the fill and surpass the gradient required to pass floods.

Simultaneously with placement of the fill, the derrick-laid section of the dam was being constructed. Rocks weighing from 3 to 10 tons each were selected from the fill and hauled to the section by cables and chains from derricks set up along the face. These rocks were set to line and others of similar size were placed behind them, the entire section being chinked up with spalls and rock hammered into a tight mass. A section through this derrick-laid rock is shown in Figure 8. It was contemplated originally that a sufficient supply of large rock would be available from the spillway excavation to build the entire derrick-laid section of the dam. However a shortage of face rock of adequate size became apparent when about half the face had been constructed. Rock in roughly 3 to 5 foot cubes was imported from the Bedford, Indiana limestone quarries for facing purpose, same being culls from the building stones quarried there.

The cutoff trench was excavated in the limestone rock to a level of seemably hard and tight rock in accordance with the plans as depicted in Figure 9. The trench was filled with concrete to the upstream face line of the dam. Reinforcing steel was embedded in the cutoff trench to extend into the concrete diaphragm which was poured directly on the derrick-laid stone. Grout pipes were also left in place. Figure 10 shows the west side cutoff trench excavation, initial top form, reinforcing steel and grout pipes.

A top form of 3-inch Tongue and Groove timber was used for placement of this reinforced concrete diaphragm. It was anchored and left in place between the cutoff wall and elevation 670, the estimated minimum level of the drawdown. Above elevation 670 the timber was removed after concrete had cured. Reinforcing steel in this slab was 1/2 of 1% in each direction. Figure 11 shows details of the concrete diaphragm, reinforcing steel and timber sheeting. Concreting operations for this face slab are shown in Figure 12.

Vertical recesses were built into the derrick-laid rock, 36 inches wide by 18 inches deep, and 48 feet on centers as depicted in Figure 13. These were filled with concrete and include an 18 inch rib protruding upward the depth of the face slab. These ribs were heavily reinforced and equipped with expandable copper water stops extended into the slab with 1 inch cork board between the rib and the slab and with tarred felt over the bearing surface of the sleeper. Horizontal expansion joints were similarly built at elevations 550, 600, 650 and 700 as shown in Figure 14. These consisted of recesses in the rock 30 inches wide by 16 inches deep, filled with concrete, well reinforced and with a tarred felt over the sleeper over which the face slab rides. A copper seal connects the face slabs and 1 inch cork board separates the seal. Concrete slabs were therefore 48 feet by 71 feet of varying thicknesses and waterstop joints between the ribs and the slabs were of 18 gage copper sheets. Figure 15 shows the recesses for the vertical and horizontal joints in the derrick-laid masonry.

FIGURE 8
Section through derrick laid rock masonry

FIGURE 10
Cutoff trench concreting operations

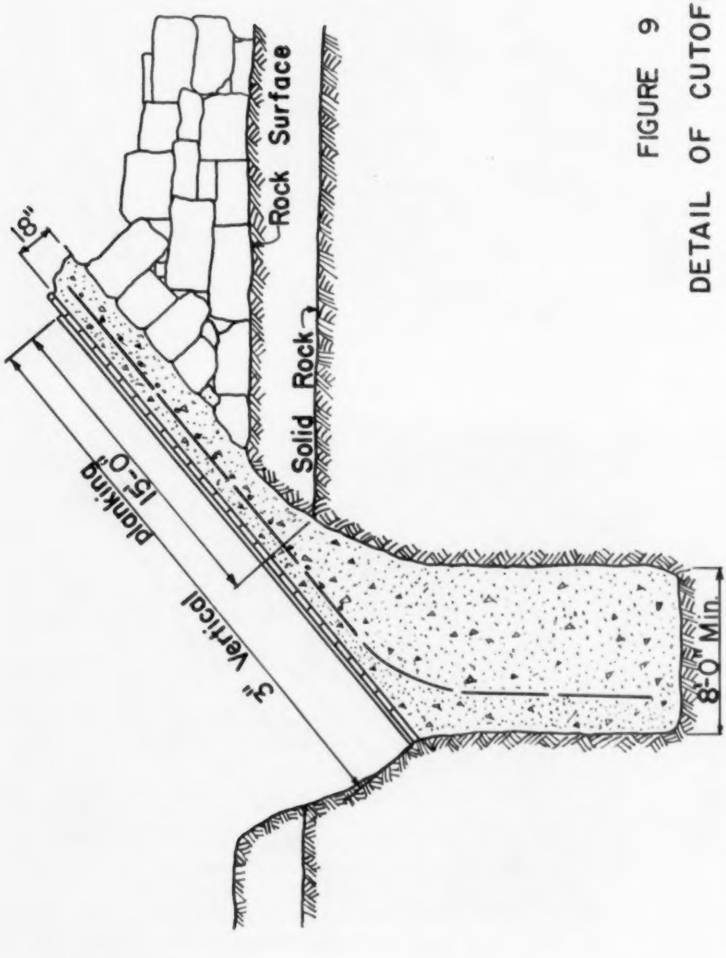


FIGURE 9

DETAIL OF CUTOFF WALL

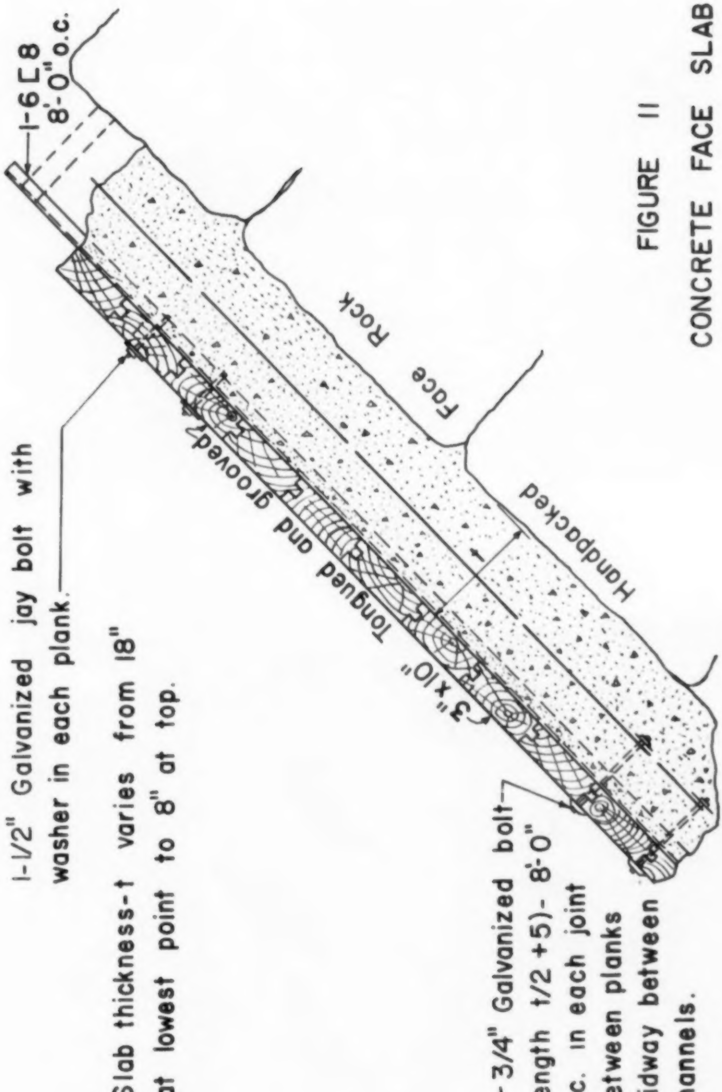


FIGURE 11
CONCRETE FACE SLAB

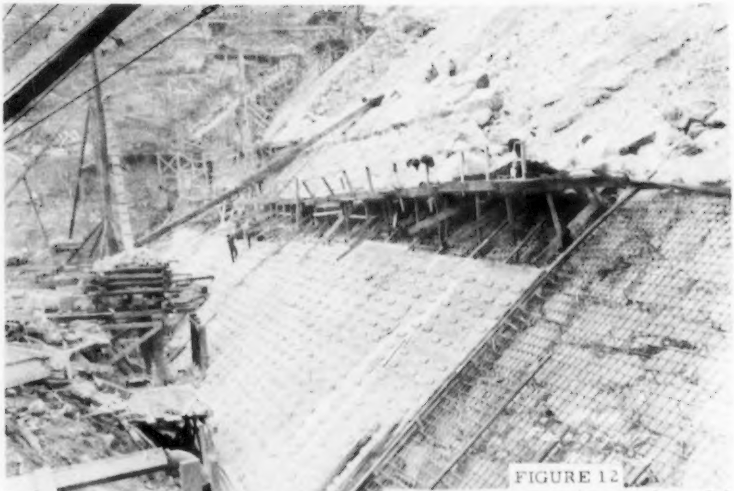


FIGURE 12
Concreting operations on face slab

FIGURE 13
Derrick laid masonry on fill
Note horizontal and vertical
joint recesses

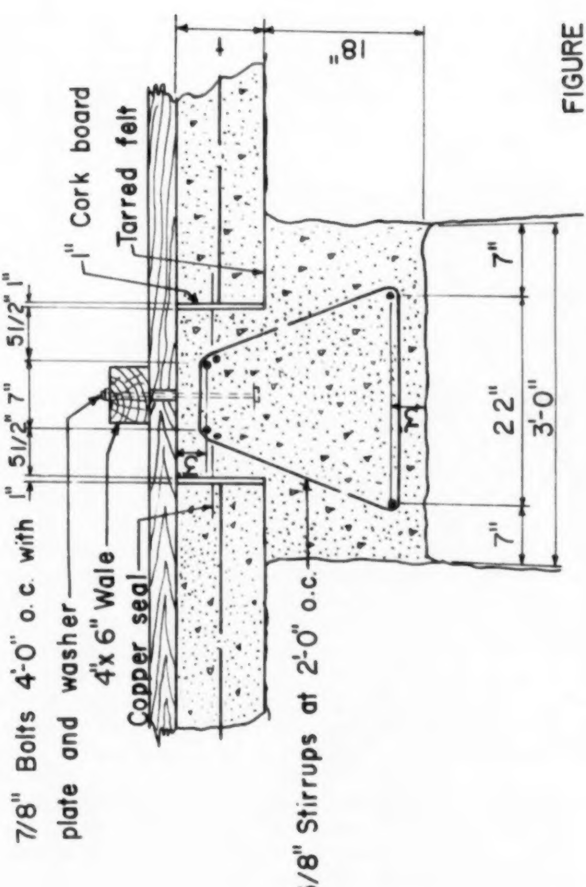
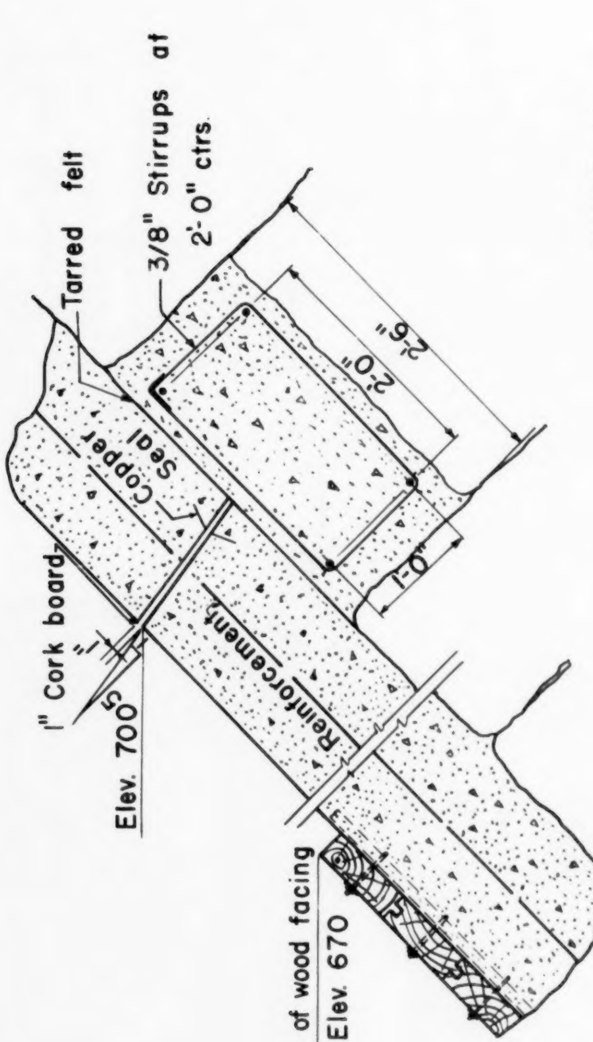


FIGURE 13

DETAIL OF VERTICAL EXPANSION JOINT



Joint shown is at elev. 700. Planking is continuous over other joints.

FIGURE 14
DETAIL OF HORIZONTAL EXPANSION JOINT

Foundation Treatment

The site is geologically situated in the Inner Blue Grass Region. The Dix and Kentucky Rivers have cut deep narrow gorges in lime-stone and shales of the Black River Ages. The oldest Black River beds are exposed in the bed of the river and for a distance of about 50 to 75 feet up the side of the gorge. The unit consists of pure, thin-bedded, dove-colored limestones with thin interlayers of calcareous shale and two thin layers of volcanic clay or bentonite. Younger Black River rocks are present in the higher parts of the abutments and overlying Trenton rocks are beneath the surrounding upland. This unit is composed of light to dark blue-gray limestone with thin interbeds of gray shale.

Structurally the rocks are generally flat lying but with a gentle southward regional dip and only local open folds break this general trend. Two sets of joints are prominent which tend to divide the bedrock into cube-like blocks. Both sets of joints have nearly vertical dips. The north-south trending set is the most conspicuous at the damsite since its trend is approximately parallel to the river and the individual joints are longer than the east-west trending set.

Weathering along such structural features as bedding planes and joints are the only defects in the bedrock at the site. The limestones at the site have been exposed to weathering for a long time and therefore a system of bedding plane seams and open vertical joints has been developed to great depths in the abutments and to moderate depths in the river bed. Most of these seams and channels are largely filled with a residual cave clay left after the solution of the impure shaly limestones and it is likely that the clay filling has washed out of many of these seams.

Because the foundation rock was limestone susceptible to solution channels and cavitation, a drilling and grouting program was pursued. Holes were 3 inches in diameter and 8868.4 feet were drilled. These were grouted using 4794 cubic feet of grout. By present day standards this program was probably insufficient but it certainly was a substantially improved operation at the time. Shot drills, which were slow and cumbersome, were used. Thirty-one (31) holes 40 feet on centers were drilled averaging 215.3 feet deep and grouted with air pressure type grout machines. Twelve (12) intermediate holes, 20 feet on centers with the primary holes, averaging 109.1 feet were drilled and grouted. Eleven (11) other intermediate holes bringing the spacing to 10 feet and averaging 71 feet deep and 3 five foot center holes averaging 34.4 feet deep were placed in areas of large grout takes in the primary and secondary holes in order to build up a grout curtain throughout. All holes were drilled under the cutoff trench and under the spillway structure. Certain horizontal seams were found and these largely determined the depths of the intermediate holes.

Because of the number of open crevices, grouts made up from ordinary concrete sand and cement were used in some cases and it is probable that these grouts often did not fill the openings completely as was intended. It is also likely that a certain amount of segregation was experienced. Grouting procedure was to run the first 50 bags of cement with mixes of 1/2 cubic foot of cement to 15 1/2 gallons of water. The next 50 bags of cement were run in mixes of one cubic foot of cement per 15 1/2 gallons of water. Thereafter sand was added in mixes 1 cubic foot of cement to 1/2 cubic foot of sand and finally 1:1 until the hole closed off. Holes were grouted from a

single setting at the top of the hole. A diagram of the original grout holes is shown in Figure 16.

Leakage

With the fill at elevation 774, derrick-laid rock at elevation 634 and above, and the concrete apron at elevation 634, the gate in the intake tower to the diversion tunnel, Figure 17, was closed and water from a general rain was stored for initial operations. Figure 18 shows the project just before closure. Shortly thereafter certain leakage was found to exist at the downstream toe of the dam near the river level. The water was generally clear and presented no apprehensions insofar as dam stability was concerned. This leak still persists even some 33 years later. As the water level in the reservoir was raised certain other leaks have also developed along the east and west banks and one at the downstream end of the tunnel.

At various times between 1925 and 1944 drilling and grouting operations were undertaken, using cement grouts, and some asphalt grout on a small scale. The sources of leakage were explored with audiophone equipment and dyes. One source of substantial leakage was indicated at a comparatively low level near the intake tower by the sonic device. Other sources were found along the face of the dam at the joints of the slabs and the ribs. Large quantities of cinders were dumped on the face and in the corner near the intake in 1936 and in a measure showed a marked temporary improvement in the leakage,

However by 1944 the main leak, measured by a weir adjacent to the powerhouse, showed leakage to be 82 cfs under 235 feet of head. Other leaks under 212 foot head were measured coming from the tunnel and east and west banks. These were 29 cfs, 8.5 cfs and approximately 15 cfs respectively. Sources of this leakage had been traced to some areas along the east cliff near the intake tower and, although not firmly established, there was evidence that some leakage originated from the west bank and through the apron of the dam.

At the same time (1944) it was decided to attack the foundation leakage problem in orderly fashion and several patterns of holes were laid out in the spillway area for the purpose of establishing possible leakage paths through the west abutment. These holes were drilled to levels below the riverbed and dye tested for leakage. Some of them showed leakage connections, the dye showing along the west bank of the river downstream from the powerhouse and at the weir. These holes were grouted with cement and asphalt as applicable.

Another pattern of holes was laid out on the east bank, drilled, tested and similarly grouted. A few of these holes showed leakage to the weir and to points along the east cliff at low levels. Measurements were made at the weir, at intermediate places between the weir and a riffle approximately 1200 feet downstream, at this riffle, at intermediate places between the riffle and a cableway measuring station approximately 1800 feet downstream from the riffle and at the cableway station. Records of the leakage measurements by dates and existing heads were made and the percentages of decreased leakage at all of these stations for different increments of head were computed. For heads of from 220-230 feet, the average decrease was 20% as a result of work done during the period between 1944 and 1956. For heads between 210-220 feet, the decrease was 29%. For heads 200-210 feet, leakage was reduced by 18% and for heads between 184-200 feet the decrease was 38%. In practically

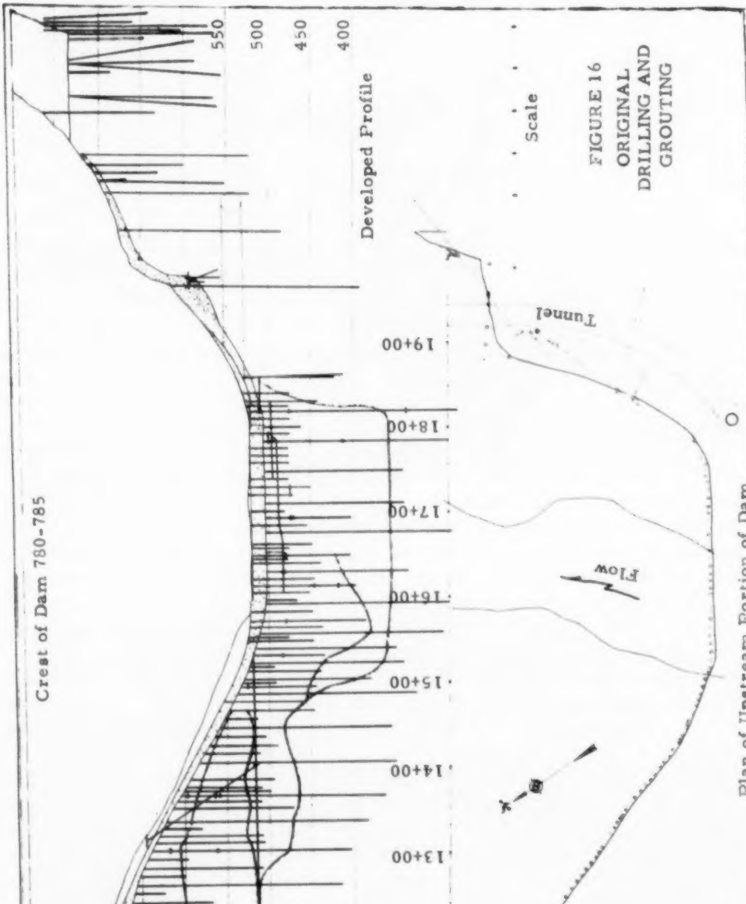


FIGURE 16
ORIGINAL
DRILLING AND
GROUTING

FIGURE 17
Intake tower closure gate
Note east side cutoff wall

FIGURE 18
Upstream view of dam just before closure

each case the extremes were not too far removed from the norms. The following tabulation is self explanatory.

Percentage Reduction in Leakage 1944-1957

Head Differential in Feet	Measured at Weir	Accounted for at Riffle	Measured at Riffle	Accounted for at Cableway	Measured at Cableway	Average
220-230	20	25	10	31	15	20
210-220	29	30	-	32	35	29
200-210	15	21	18	25	12	18
184-200	40	40	33	39	40	38

The only continuous leakage measurement over a range of heads is available at the weir which is also the predominant leak. A graph showing the effect of the drilling and grouting work is shown in Figure 19.

As this work progressed it became obvious that some type of maintenance program should be established to arrest this leakage. Even if the leakage could not be completely eliminated at once a continuing grouting program should be successful in the ultimate correction of this leakage and in the meantime its effectiveness in reducing the leakage or at least keeping it from increasing was recognized. This was done and the work continues. The results are at times very positive - at other times they indicate only that a holding action has been established. In any case, the effort and the expenditures seem amply justified because an examination of the records and results indicates that without this work the leakage was prone to have increased by greater proportions than the reductions actually affected. The effectiveness can therefore not be measured accurately in dollars and cents but the need for the work is an established fact.

In 1944 certain leakage was coming from the tunnel. This increased year by year until the leakage at the downstream tunnel portal had reached such proportions in 1949 that it was decided to undertake some remedial work at this end of the tunnel, working downstream from the penstock plug. The work had hardly begun when the water under pressure broke out a portion of the rock below the invert and to the east of the concrete tunnel lining just upstream from the concrete plug that holds the penstocks. It was necessary to close the intake tower gate and to discontinue operations while this major repair was being made. However leakage in this location had no bearing on the other leakage correction work except that the leak developed in a bentonite seam near the invert on the east side of the tunnel just upstream from the penstock plug.

This repair consisted of clearing out all debris, rocks and other accumulations, replacing the lining and filling the cavity with concrete. Furthermore the plug was lengthened 29 feet upstream with formed concrete penstocks in this new section connecting to the existing steel penstocks. The entire plug section, now 64 feet long was then drilled radially with holes 30 feet into rock, 16-18 holes being spaced approximately equally around the periphery of the tunnel and 7-10 feet on centers longitudinally. These holes were drilled from inside the penstocks and grouted to refusal with neat cement grout under 60#/sq. in. pressure immediately after each was drilled. A total of 119 holes were drilled to saturate a circle 84 feet in diameter from the centerline of the tunnel, and 2360 cubic feet of grout were used for this operation.

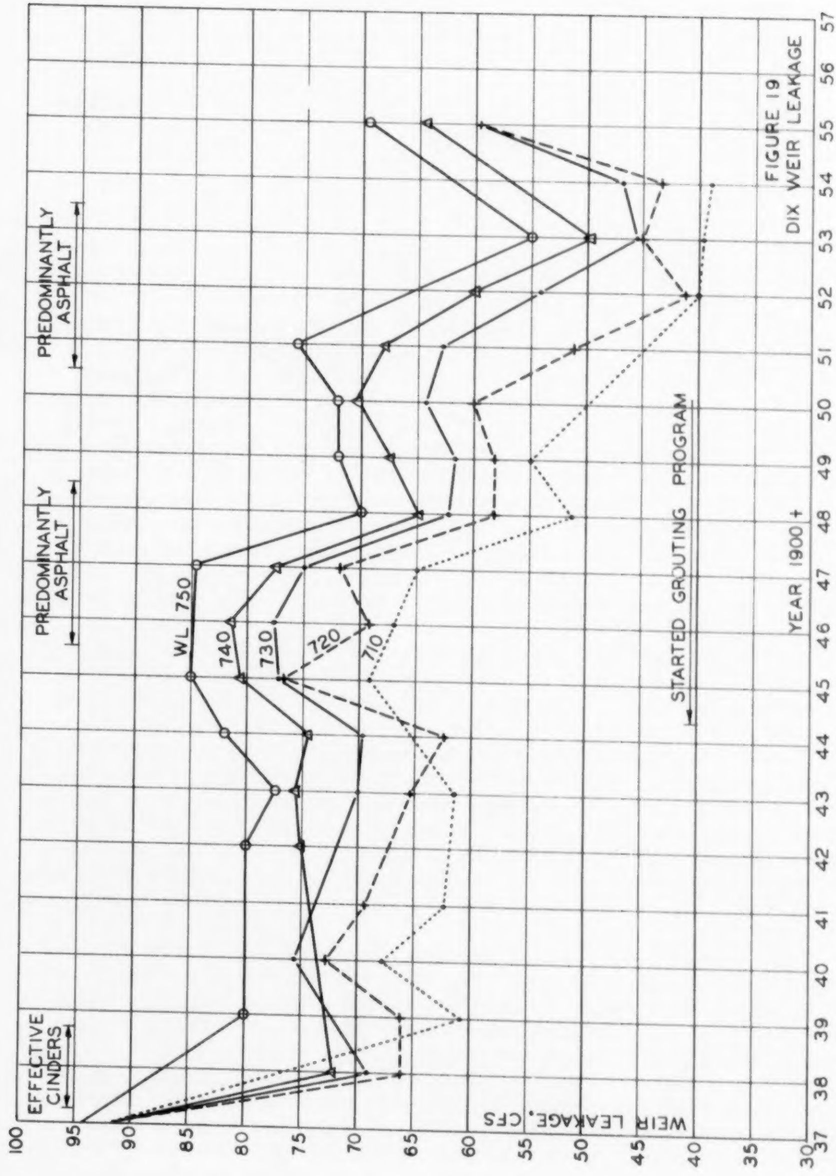


FIGURE 19
DIX WEIR LEAKAGE

The repair has been highly satisfactory and since it was completed there has been no leakage at the downstream tunnel portal.

Settlement

Figure 20 shows the derrick-laid section and the concrete apron nearing completion to the crest of the dam. At the time of construction a series of copper pins was established in the face concrete at the crest and at elevation 725 to record settlement of the dam. From time to time readings of the elevations and horizontal positions of these pins were taken. A limited number of readings could be obtained at elevation 725 due to the water level in the reservoir.

The significant point about settlement and lateral movement of the fill downstream is that it is still taking place after 32 years at both the crest and at elevation 725. Although there exist a few inconsistencies in the settlement readings, the trend is uniformly along parabolic curves. The inconsistencies are of such a minor nature as to be readily accounted for in spot readings of the field measurements.

An upstream elevation of the dam and actual elevations at the crest are shown for various years in Figure 21. This figure also shows the relative vertical settlement at four typical stations for readings taken both at the crest and at Elevation 725. Differential settlement and horizontal displacement at the crest are shown in Figure 22. The readings that could be obtained to show vertical settlement and horizontal movement at Elevation 725 are shown in Figure 23 which also shows the relative horizontal displacement of the dam at two typical sections for readings taken at the crest and at Elevation 725.

Some general deductions may be made from the settlement records.

1. As expected, the greatest settlement (and downstream movement) has taken place in the deepest sections. At the crest measuring stations, a maximum vertical settlement was 4.22 feet from 1925 to 1957 or 1.57%. The maximum downstream displacement at this level is 3.19 feet during the same period, or 75.5% of the vertical settlement.
2. Judging from the shape of the settlement curves, additional settlement in reduced increments will continue over the next 10 to 20 years. Having superelevated the dam by 5 feet in the deepest section when it was constructed was therefore a very good estimate of the settlement expectancy.
3. Settlement and movement downstream at the 725 level is less than at the crest (785-780) as was to be expected. Also incremental settlement and downstream displacement are less than at the crest after the 32 years, another anticipated feature. Maximum vertical settlement at this level during this period of time is 3.60 feet or 1.71% and corresponding maximum horizontal displacement is 2.60 feet, or 72% of the vertical settlement.

In general it can be said that the settlement and horizontal displacement behaviors are rational although extending over a much longer period of time than what perhaps was originally envisioned. However since the original superelevation was 1.85% of the height of the dam, it will still be some time before the settlement catches up with the depth of settlement originally provided for it.

FIGURE 20
Project nearing completion

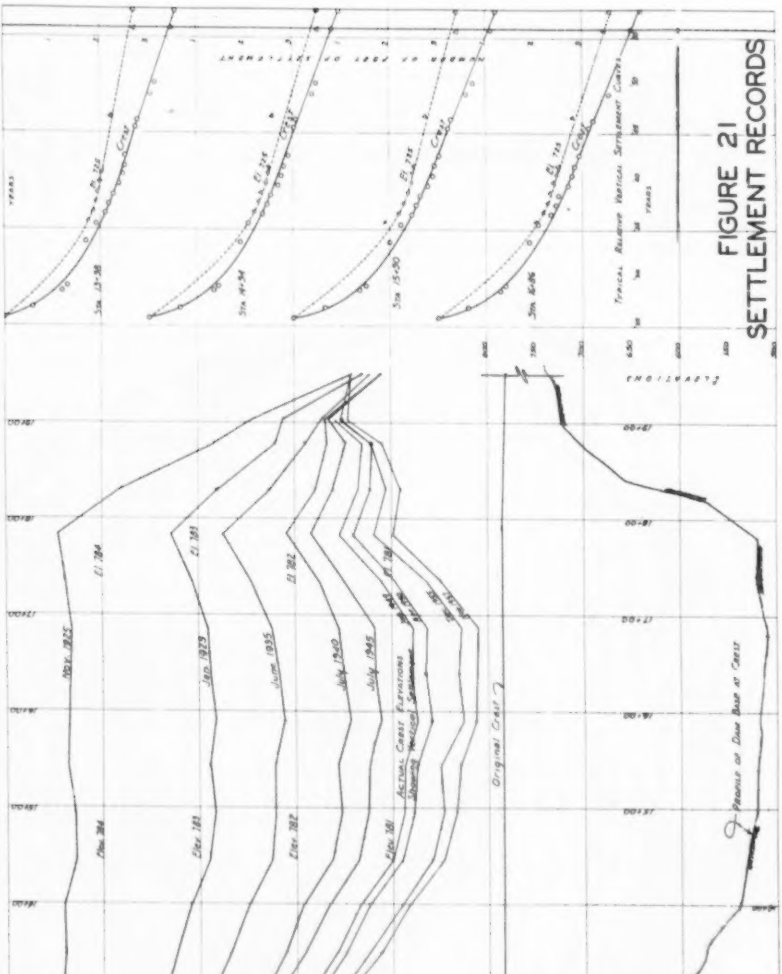


FIGURE 21
SETTLEMENT RECORDS

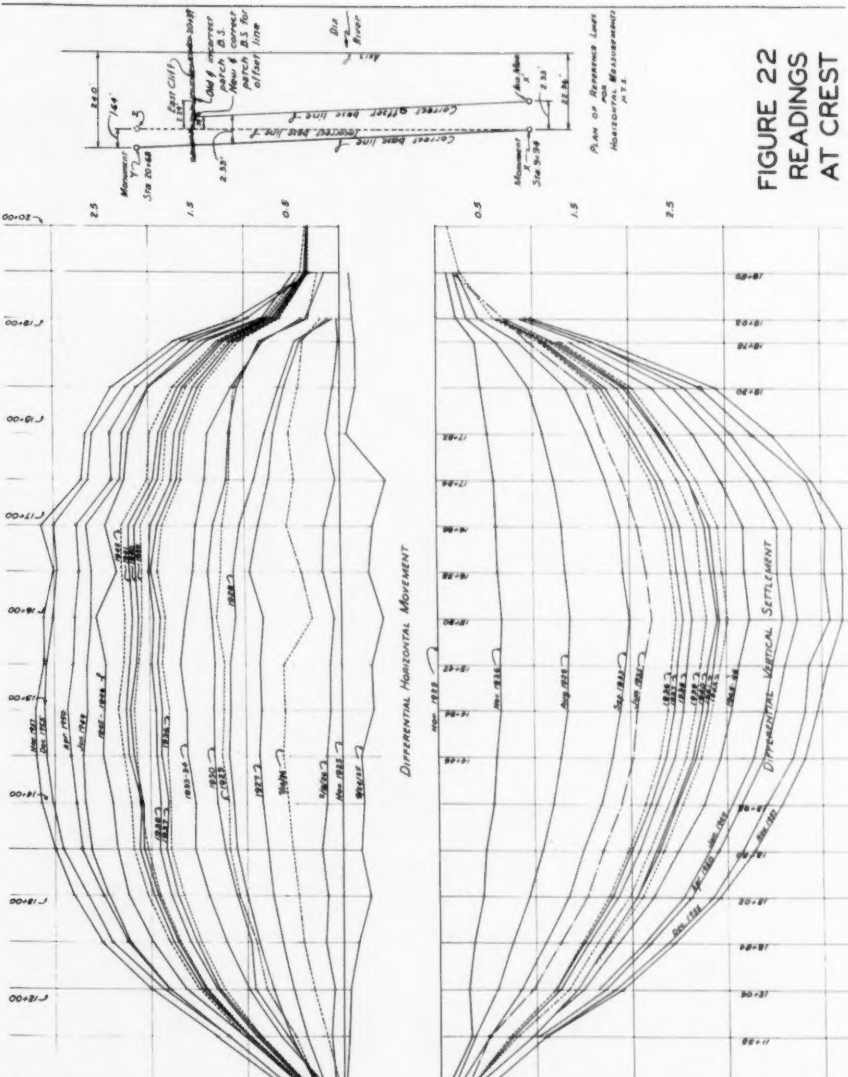


FIGURE 22
READINGS
AT CREST

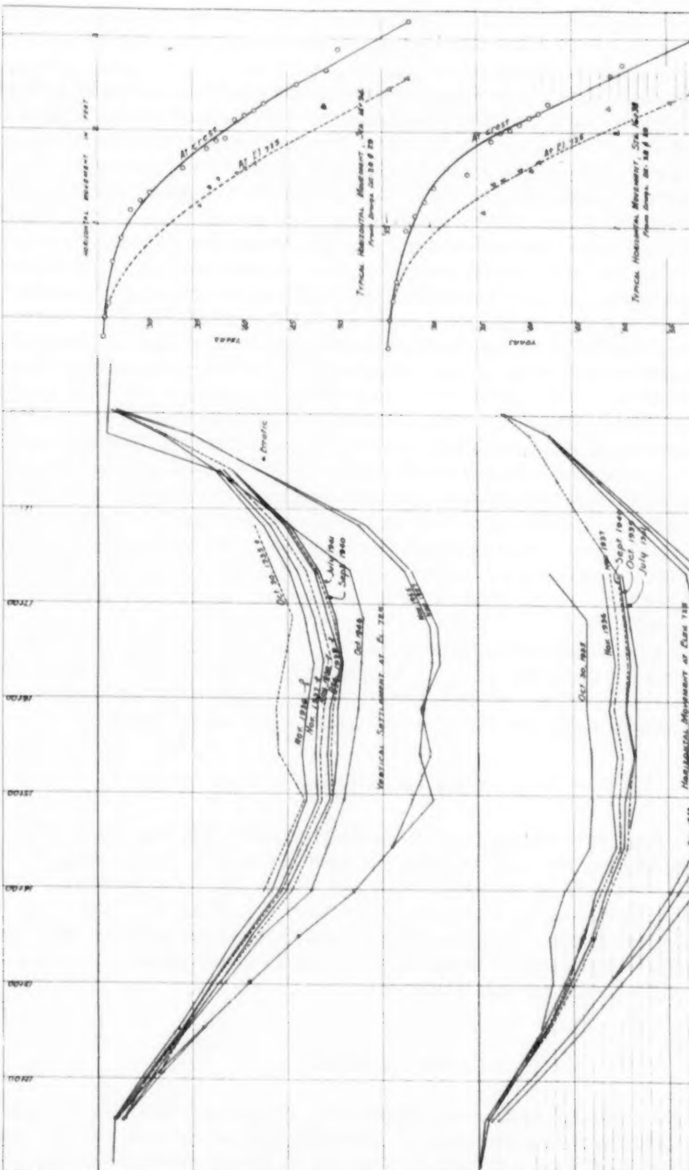


FIGURE 23
READINGS AT ELEV. 725

Face Slab and Derrick-Laid Masonry

There is no way to ascertain the condition of the concrete face slab below low water level, elevation 670, without draining the reservoir. Similarly no inspection of the condition of the derrick-laid masonry section is possible because it is covered by the face slab.

In recent years the lowest water level attained has been elevation 696 in the fall of 1946 and between September and December of that year, detailed inspections of the face slab and particularly the vertical and horizontal joints were made between elevations 725 to 696 as the water level dropped.

The results of this inspection indicated numerous breaks in the vertical and 700 horizontal joints. The copper seals were pulled loose in a number of places. The concrete ribs forming the vertical joints were broken in a number of cases at the lower levels and there was audible leakage below elevation 696 in a substantial number of joints. Also there were quite a few cracks in the face slab varying from hairline cracks to open cracks, the latter occurring along the east cliff where vertical movement is augmented by horizontal movement both downstream and along the axis due to the steepness of the east cliff.

The greatest significance of this investigation was that the face condition was more seriously impaired progressively at the lower levels where water pressures and weight of the structure are greater. Conversely the upper portions of the face are exposed to greater temperature changes when exposed to the sun and the weather than those areas below water level. However the vertical and horizontal movements of the dam cause the slab to pivot around horizontal joints and the joint between the face slab and the cutoff wall so that some opening of these joints to potential leakage is not only possible but also highly probable.

While not of any immediate hazard nor an economic liability, it is recognized that some time in the future, the plant will have to be shut down and the face completely repaired. At such time a more complete and up-to-date grouting program under the cutoff trench should also be undertaken.

Reservoir Silting

Only one reservoir silting survey has been made in the Dix Reservoir. It was made in 1941 by the Conservation Service of the U. S. Department of Agriculture. Over the 16 years of reservoir life in 1941, the reservoir volume had been decreased by 1.25% or an average of 0.08 percent per year. Because only one measurement has been made, it is by no means certain that this rate will be maintained. Other measurements would be helpful in establishing the silting rate for this reservoir.

CONCLUSIONS

Built at a cost of approximately $7,000,000, the project has been an economic success, averaging an output of approximately 49 million kwh per year. Originally it was used as a base load plant but in recent years as larger steam generating stations began replacing hydro stations as base load plants, the value of this project has become even greater as a peaking station which is its present function in the system operations.

Dix Dam causes the impounding of water for hydroelectric energy purposes. Furthermore the City of Danville, Kentucky, a State Hospital and the E. W. Brown steam generating station are dependent on the lake for their water supplies and numerous fishing and recreational camps have been built on the shores of the lake. For these reasons it is not readily feasible to drain the reservoir, inspect and repair the face and do remedial foundation treatment under the cutoff trench. To that end the choice of a faced rock fill dam in a limestone region for power and other multi-purposes should probably be discouraged although for other purposes, where the reservoir can be drained periodically, or at locations where there is no danger of foundation leakage, a rock fill dam should function satisfactorily and in all likelihood can be built economically.

ACKNOWLEDGMENTS

The writer is indebted to the Kentucky Utilities Co. for permission to use the data published herein. It has been assembled by the writer over a period of years with the cooperation of Messrs. R. M. Watt, Chairman of the Board of Directors, E. W. Brown, Vice President until his death in 1957, W. A. Duncan, Vice President since 1957, Henry L. Johnson, General Superintendent of Production and Mr. G. D. Glass, Superintendent of the E. W. Brown Steam and Dix Dam Hydro Generating Stations, all of the Kentucky Utilities Co.

The writer also wishes to acknowledge the cooperation of the Harza Engineering Co., Inc. for making available prints of the original designs of the project.

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ROCKFILL DAMS: COGSWELL AND SAN GABRIEL DAMS

Paul Baumann,[1] M. ASCE
(Proc. Paper 1687)

FOREWORD

This paper is one of a group from the Symposium on Rockfill Dams, June 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element and is described approximately as follows: a dam in which at least 50% of the maximum section is quarried rock; and in which at least half of the rock is dumped from lifts rather than placed in layers. This includes the types with impervious face membranes, sloping earth cores, thin central cores and with thick cores as limited roughly by the above description.

The objective of the Symposium is to assemble up-to-date information on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Features of special interest attending the design, construction, and performance of two rockfill dams 280 and 380 feet in height, more or less, are treated in this paper. Particular attention is paid to such novel features in the design and construction as have proved to be advantageous and successful as well as those which, ingeniously, though they may have been conceived by the designer, were found to be undesirable, if not detrimental. The principal reason for the latter was the fact that conditions in the field failed to fully conform to those anticipated. Through frank discussion thereof it is hoped to avoid repetition and to thereby render a service to the profession.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1687 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 3, June, 1958.

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INTRODUCTION

The San Gabriel and Cogswell Rockfill Dams of the Los Angeles County Flood Control District, formerly and perhaps better known as San Gabriel Dams No. 1 and No. 2, serve the dual purpose of flood control and water conservation. As shown in Fig. 1 they are situated on the main trunk and the West Fork of the San Gabriel River in Los Angeles County, California, which carries runoff from the San Gabriel Mountains to the sea. Their respective drainage areas comprise 163.5 and 39.2 square miles. They have the distinction of differing by 100 feet in height and 1000 feet in elevation above sea level, in round figures. Design and construction of Cogswell Dam preceded those of San Gabriel Dam. Hence, Cogswell Dam will be treated first. Furthermore, the performance of Cogswell Dam has so far only been treated in fragmentary discussions of a paper(1) whereas the performance of San Gabriel Dam was quite completely treated by the writer.(2)

To avoid confusion the original names of these dams, namely San Gabriel Dams No. 1 and No. 2, were adhered to in preparing the exhibits accompanying the text, in that the majority thereof were taken from widely distributed specification drawings of these dams.

Performance of Cogswell Dam

Exploration

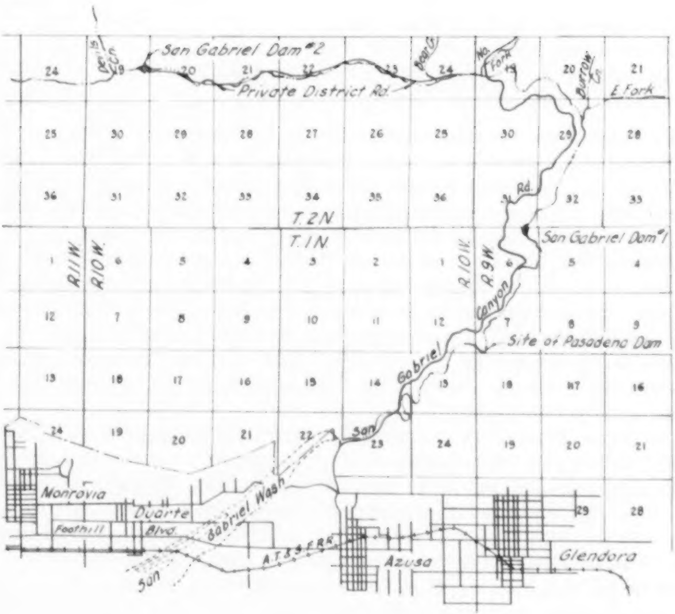
Exploration of the foundation of the proposed dam was conducted by the Flood Control District through 7 diamond drill holes, 14 tunnels, 2 shafts, and 4 trenches. Respective information was made available to bidders. Through this exploratory work the general foundation characteristics were reasonably well established. Isolated variations therefrom which had to be anticipated under existing conditions at this dam site necessarily remained to be discovered and dealt with in the course of construction.

Design and Specifications

The specifications provided the material for the rockfill to consist of three classes A, B, and C of large rock with maximum size on the downstream face and toe, and with derrick placed rock, commonly known as packed rock, immediately below the facing. The character of all rock was to be sound, hard, durable, angular quarried rock, weighing not less than 160 pounds per cubic foot (p.c.f.); to be unaffected by air and moisture and of such toughness as to withstand dumping without undue shattering or breakdown; and to have a minimum compressive strength of 5000 pounds per square inch (p.s.i.).

Class A rock, for general use throughout the main fill of the dam, was to be a well graded mixture, 40 per cent of which to vary in weight from quarry chips to 1000 pounds, 30 per cent from 1000 to 3000 pounds, and the remaining 30 per cent from 3000 to 14,000 pounds. In addition, this mixture was not to contain more than 3 per cent of its total weight in quarry dust and the maximum dimension of any piece was not to be more than three times its minimum dimension.

Class B rock was to be selected extra large rock one-half of which to weigh not less than 14,000 pounds and the other half not less than 6000 pounds each. The greatest dimension of each piece was to be not more than four times its



LOCATION MAP

1 ½ 0 1 2 mi.

FIG. 1

least dimension. This rock was to be placed at the downstream toe and on the downstream face of the dam.

Class C rock was to vary in weight from quarry chips to 14,000 pounds, the relative proportion of the various sizes to be regulated according to the requirements of placing to result in a packed rockfill of maximum density.

The source of all rock was to be a quarry one and one-half miles upstream from the dam site opened by the Flood Control District in Devils Canyon, a tributary to the West Fork of the San Gabriel River (Fig. 1). This rock had been tested and found to satisfy the above requirements. It is a granitic gneiss. Typical test results as averages of 211 tests were as follows:

Specific gravity: 2.80 or 174.7 p.c.f.
Moisture content when saturated: 0.45 per cent
Breakdown due to max. 18 feet drop: 5.04 per cent
Compressive strength: 6629 p.s.i.

Loose rockfill was to be placed in lifts of not to exceed 25 feet. It was to be spread approximately level and parallel to the axis of the dam. Placing of the fill from the abutments toward the center and thus leaving a gap at or near the center was not to be permitted. Due to scarcity of stream flow (1932), sluicing for consolidation of the loose rockfill was not provided.

Packed rockfill was to be so placed that the natural bedding planes were to be between horizontal surfaces and surfaces approximately normal to the face of the dam. Great care was to be taken in the construction of this packed rockfill and particularly in the keying into the loose rockfill below.

Stream diversion during construction of the dam was to be accomplished through the outlet tunnel shown in the plan view of Fig. 2. Hence, this tunnel was to be constructed first.

The spillway, capable of carrying the maximum flood peak of 56,000 c.f.s. that could reasonably be anticipated, was located in a cut through the right abutment of the dam as shown in the plan view of Fig. 2a and profile and sections in Figs. 2b, c and d. A reinforced concrete arch bridge, not shown in Fig. 2a, was to provide access to the dam from the main road to the south. Capacity and area curves of the reservoir are shown in Fig. 2c.

As shown in Figs. 3a, b and c, the maximum cross section, profile along center line of crest, and projected plan of upstream face of the dam, the design followed conventional features of rockfill dams except for the facing slab which was intended to have greater flexibility than the more conventional type of facing slab. Fig. 4 shows details a, b and c of this facing slab, the distinguishing features being the lamination of the slab proper, its anchorage to the sub-slab, and attachment to the floater slab which in turn was hinged to the cutoff wall. Through this arrangement, it was hoped to provide sufficient flexibility for the slab to adjust itself to residual settlement of the dam over a period of years of its initial performance.

The salient provisions were as follows: A sub-slab placed directly on the packed rock in 30-foot squares with six-inch open joints between the squares; anchor bars set in the packed rock protruding through the sub-slab and serving as anchorage for the laminated facing; the number of laminations of the facing varying between five at the toe of the dam for concrete, or four for shotcrete (gunite) to two at the crest for either concrete or shotcrete; recesses in the packed rock provided at points of anchorage which were made an integral part of the sub-slab in form of spurs to prevent slippage.

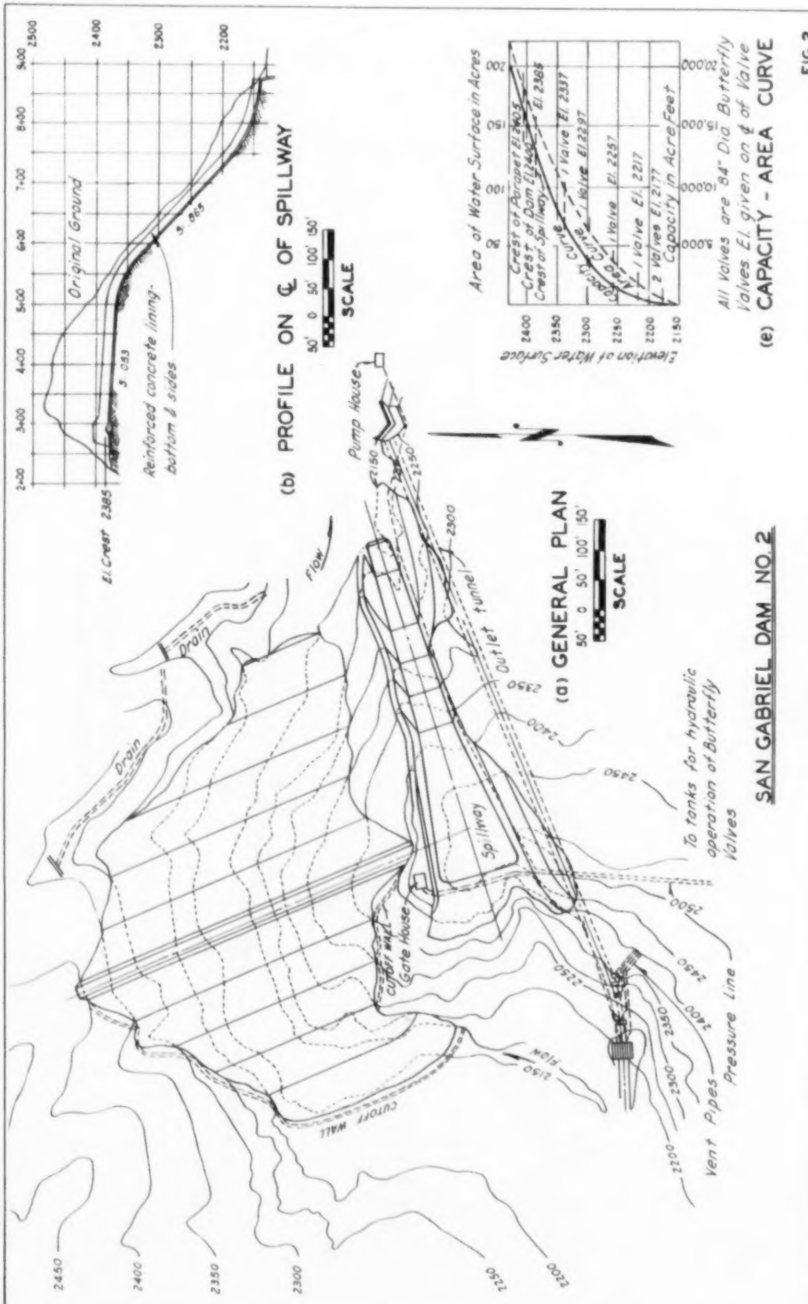
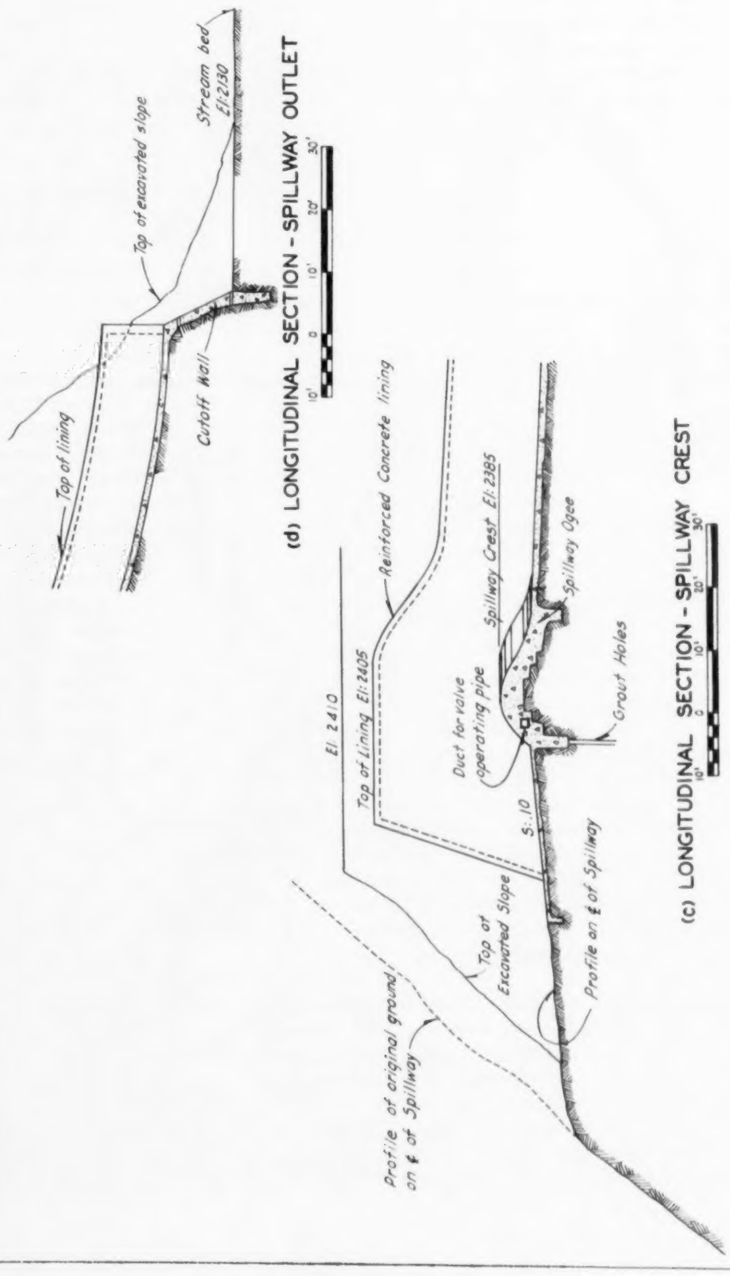

SAN GABRIEL DAM NO. 2
(a) GENERAL PLAN
SCALE
50' 0 50' 100' 150'
Drain
Flow
Outlet tunnel
Spillway
Pump House
Gate House
Cutoff wall
Cutoff wall
Vent Pipes
Pressure Line
To tanks for hydraulic operation of Butterfly Valves
(b) PROFILE ON ℄ OF SPILLWAY
SCALE
50' 0 50' 100' 150'
Original Ground
El. Crest 2385
Reinforced concrete lining-bottom & sides
(e) CAPACITY - AREA CURVE
Area of Water Surface in Acres
Elevation of Water Surface
Capacity in Acre Feet
Crest of Parapet El. 2405
Crest of Dam El. 2400
Crest of Spillway El. 2385
All Valves are 84" Dia. Butterfly Valves El. given on ℄ of Valve

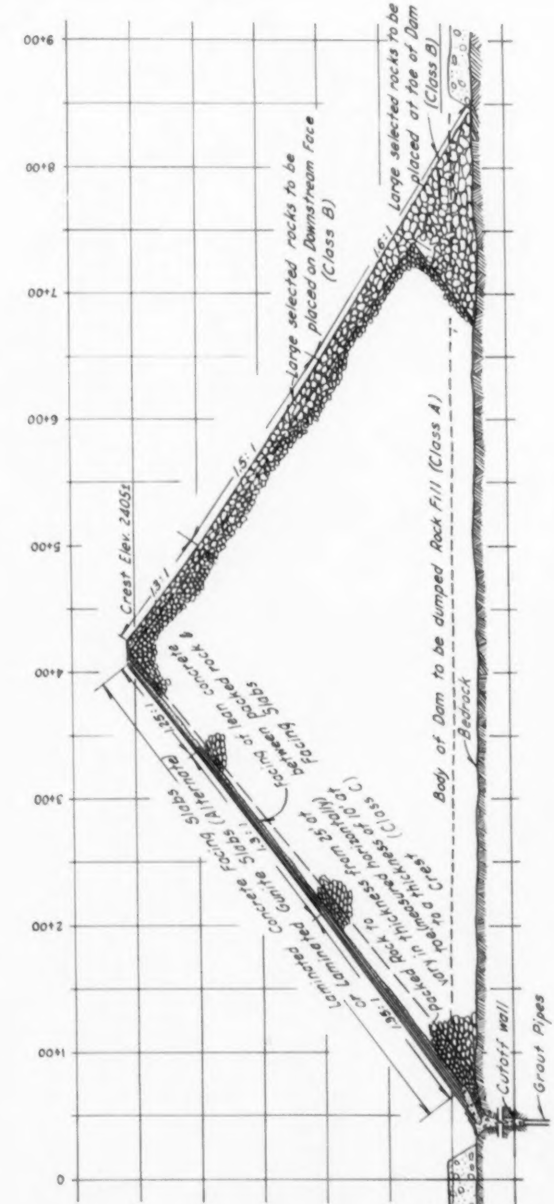
FIG. 2



(d) LONGITUDINAL SECTION - SPILLWAY OUTLET

(c) LONGITUDINAL SECTION - SPILLWAY CREST

FIG. 2

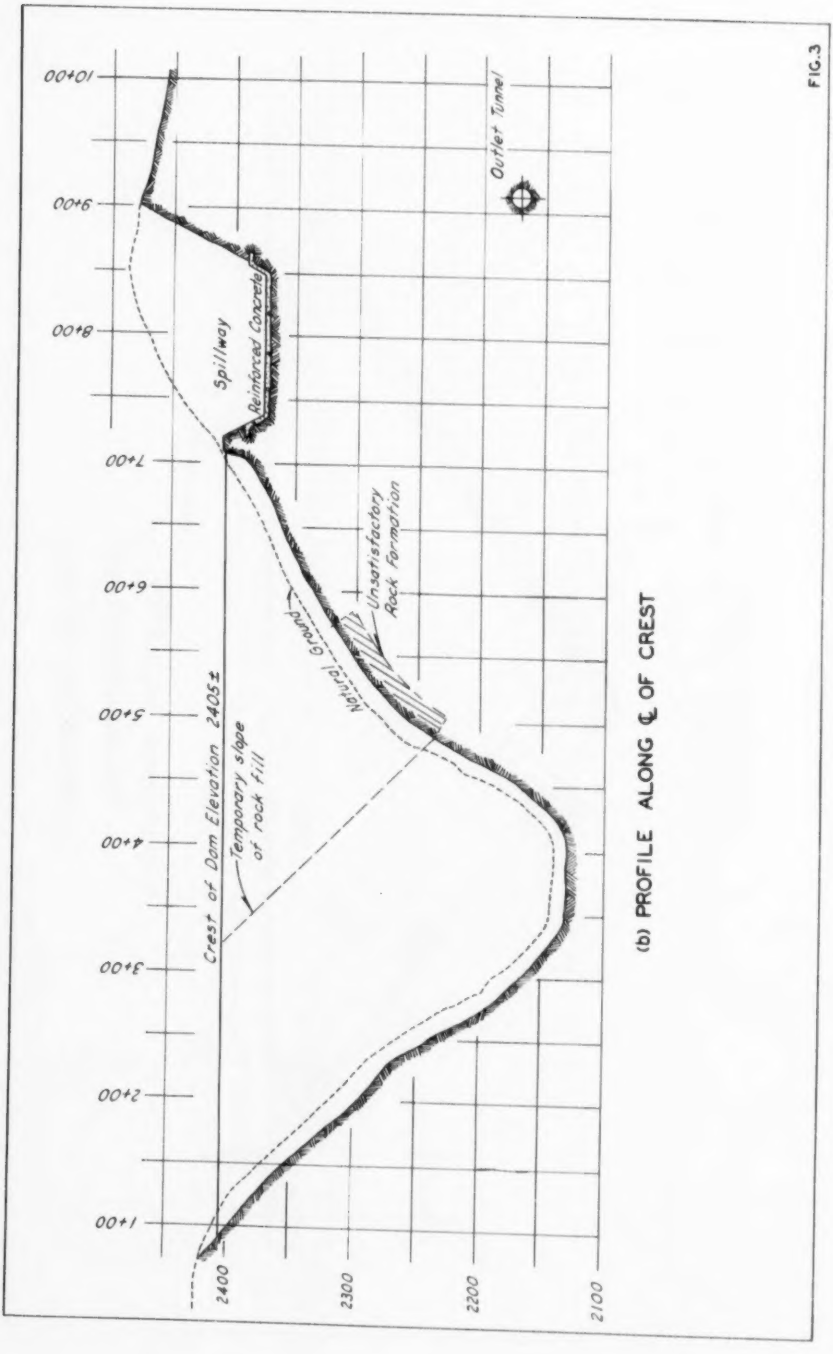


(a) MAXIMUM SECTION OF DAM

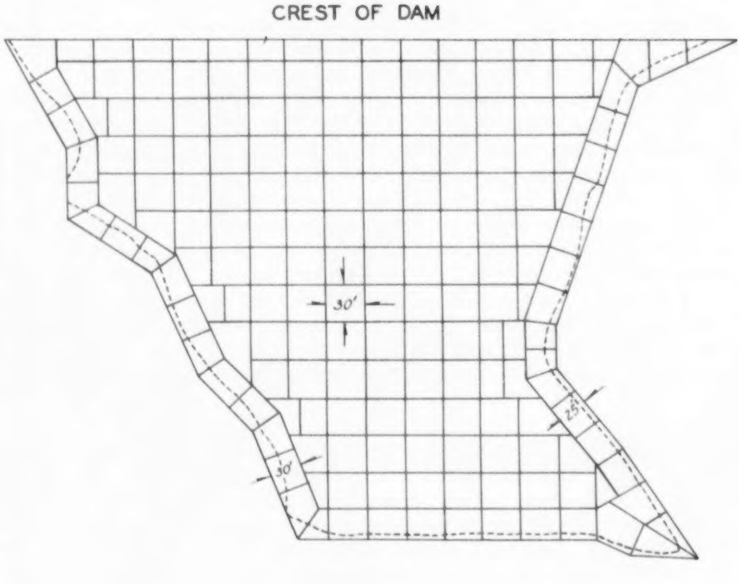
50' 0 50' 100' 150'

SAN GABRIEL DAM NO. 2

FIG.3

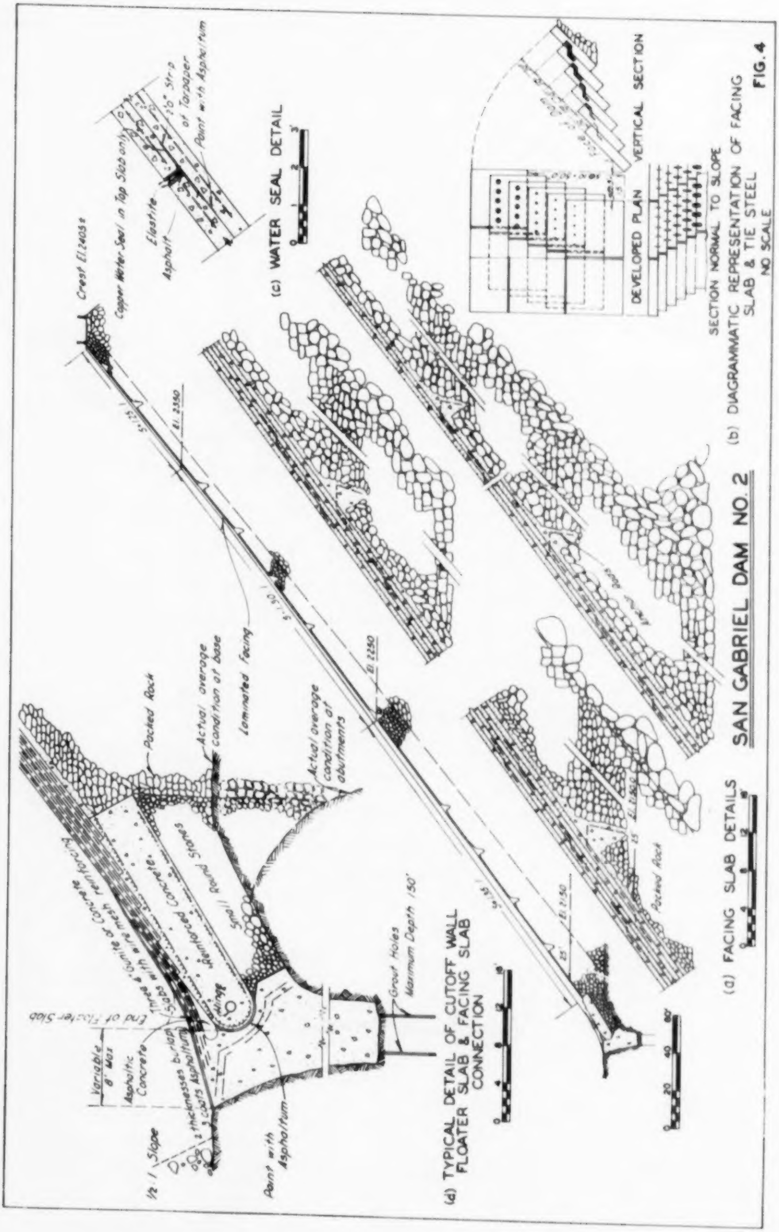


(b) PROFILE ALONG ℄ OF CREST



(c) PROJECTED PLAN OF UPSTREAM FACE

FIG. 3



SAN GABRIEL DAM NO. 2

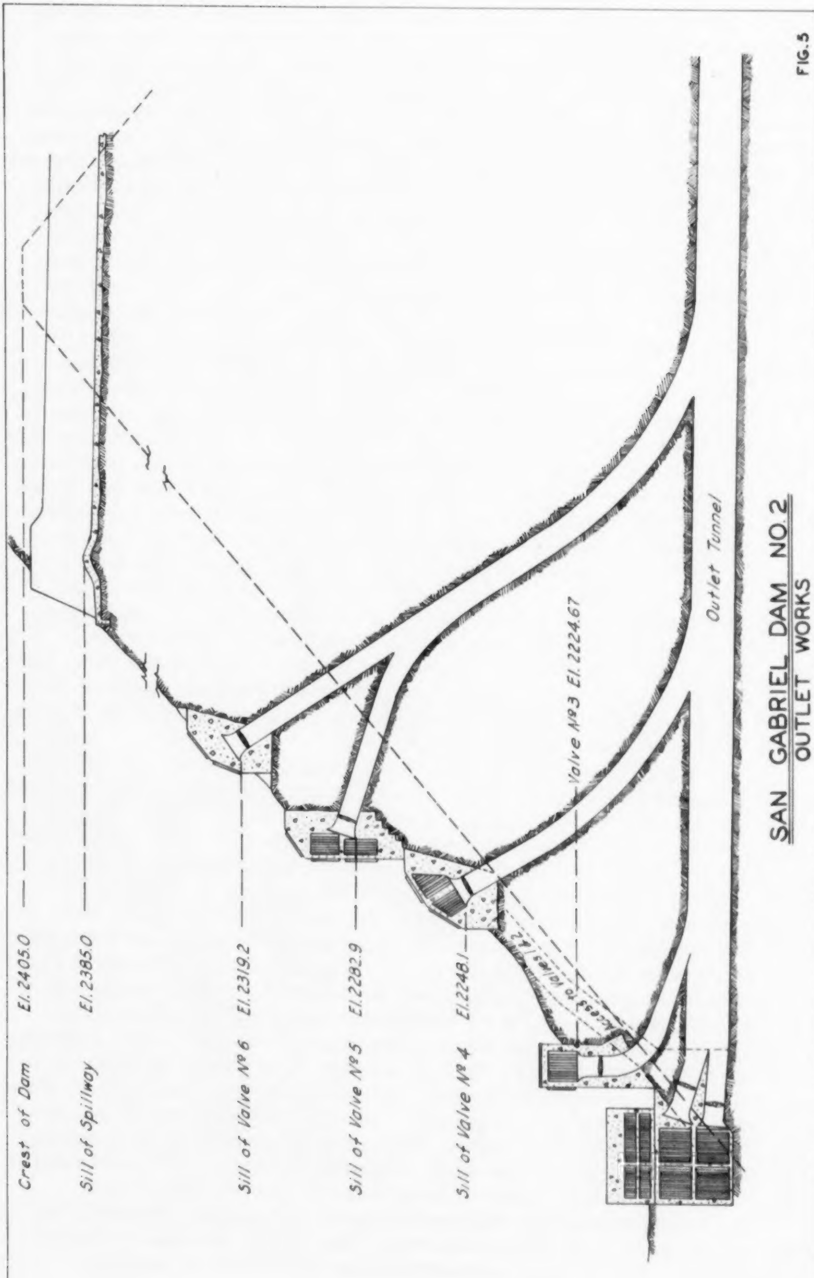
FIG. 4

Perhaps the most novel feature was the floater slab hinged to the cutoff wall as shown in detail in Fig. 4d. It is obvious that for this floater slab to rotate around the hinge which was formed by short lengths of 10 in. extra heavy pipe, the rock underneath described as "small round stone" (under Class A rock) would have to yield. Furthermore, such yield would have to be proportional to the distance from the hinge so as to create uniformly distributed bearing pressure on the floater slab and thereby avoid critical bending stresses. While this requirement should reasonably be satisfied along the base of the dam, along the abutment it was problematical because of the unavoidable shattering and sloughing of the rock due to blasting in connection with benching and trenching for the construction of the cutoff wall. Hence, instead of the ideal rock section at the base as shown in Fig. 4d (solid line), the actual section at the abutments could readily turn out to conform to the one indicated by a dash line. Therefore, the small round stone fill supporting the floater slab next to the cutoff wall could act as a fulcrum due to its shallow depth as compared to the much greater depth with increasing distance toward the rock face from the cutoff wall and therefore from the hinge. For small residual settlement and deflection of the rockfill, especially as a result of full water pressure on the facing, this fulcrum effect could have been relatively harmless and the stresses in the floater slab could well have been within allowable limits. However, as a result of major settlement and deflection, the floater slab would be unable to rotate freely and to follow the deflection of the facing because of the fulcrum support. This could only lead to over-stressing and cracking at the top of the floater slab near the hinge.

As also shown in Fig. 3a, the design provided for the excavation of the stream bed materials and for the placing of the rockfill on bedrock along all of the contact surfaces of the dam. These contact surfaces were to be stripped to rock of sufficient strength to support the rockfill of the dam. The cutoff wall along the entire contact line of the face of the dam was to be carried to sound rock and was to be grouted along its entire length so as to form an effective water seal. (See Fig. 4)

The outlet works, shown in Fig. 5 were arranged in form of circular, inclined shafts varying between 7'-0'' and 8'-6'' net inside diameter connected to the circular outlet tunnel, 14 feet in net inside diameter, which, as previously mentioned, served as diversion tunnel during construction. Each of the four inclined shafts was to be controlled by an 84-inch butterfly valve, protected by a cage-type trash rack with 6-inch bar spacing. In addition, two 84-inch butterfly valves were provided at the upstream portal of the main tunnel likewise protected by a cage-type trash rack structure. All butterfly valves were to be hydraulically controlled from the gate house shown in Fig. 2a. (Actually the gate control house was located on the right or south side of the spillway.) Water supply of adequate pressure for the operation of the valves was provided through a storage tank above the south or right abutment at elevation 3230 with water surface at elevation 3236. Thus, the water pressure at the lowest butterfly valve was to be 459 p.s.i. For the supply of water to the tank, a pump station near the lower portal of the main tunnel was provided. Vents for all butterfly valves were likewise provided (Fig. 2) through pipes anchored to the cliff of the right abutment with the air intake structure at elev. 2408.5 or 10.5 feet above highest water level of the reservoir.

To check qualitatively the complex hydraulic computations attending the outlet works, model tests were performed. The model, was made of transparent, plastic material which permitted close observation of possible



SAN GABRIEL DAM NO. 2
OUTLET WORKS

FIG. 5

cavitation or other undesirable features under various operating conditions.

Construction

Construction of Cogswell Dam started in March, 1932 and progressed according to schedule until the Spring of 1933. At this time, difficulties were encountered in the finding of satisfactory rock at which to terminate the cutoff wall approximately between elevs. 2235 and 2320 of the south or right abutment. This distance is indicated in Fig. 3b. Further placing of rockfill against this reach of the abutment was halted by the California State Engineer pending additional deepening of the cutoff trench in quest of satisfactory rock. Meanwhile, however, the placing of rockfill continued along the left abutment, leading to a slope, corresponding to the angle of repose of the dry rock as indicated by a dash line in Fig. 3b. Hence, the very condition was thereby created which, according to the specifications, was to be avoided.

After further deepening of the cutoff trench did not reveal satisfactory rock and the fill at and adjacent to the north or left abutment had reached the crest of the dam, it was decided to proceed with the pouring of the cutoff wall in the reach in question and to subsequently complete its base through tunneling and stoping. This arrangement made it possible to fill the V-shaped gap by dumping the rock from the completed level, thus initially from a much greater lift than the maximum 25 feet provided in the specifications. Under these conditions, arching between the fill slope and the abutment wall was unavoidable.

The contractor chose the use of shotcrete (gunite) as an alternate to concrete for the sub-slab and the laminated facing as well as for the lining of the inclined shafts of the outlet works. Construction of the facing followed the placing of the rockfill and the packed rock part thereof, with a minimum of necessary lag and by December of 1933 had been about 80 per cent completed.

On December 31, a major storm swept in from the Pacific Ocean which by noon of January 1, 1934 had yielded 15.07 inches of rain at the dam. The application of this natural lubrication to the dry rockfill caused the latter to settle, especially that part dumped in the gap at the right abutment. Immediate settlement there amounted to some 4 per cent. Vertical settlement was accompanied by slight bulging in the upstream lower half of the dam, resulting in damage to the laminated facing and the sub-slab, particularly near the abutments. The packed rock section which in effect was an inclined, dry rock wall resting on the dumped rockfill below, could not follow the settlement of the latter because of its greater rigidity. Consequently, the packed rock section buckled, thereby adding to the destruction of the facing to that which would have occurred had the settlement been entirely plastic.

Pending appraisal of the damage and preparation of remedial measures, the contract work came to a standstill.

The storm reversed the water supply from deficient to abundant. Thus, water in ample volume was now available for artificial sluicing of the rockfill. This was accomplished by drilling holes through facing and sub-slab, setting pipe with fire hose connectors into these holes, and pumping clear water into the rockfill to hasten final settlement. After several months of this sluicing operation, the afore-mentioned, maximum settlement of 4 per cent had been increased to 6 per cent. The average residual settlement amounted to 4.5 per cent. A nearly stable condition was indicated so far as the influence of dead weight was concerned. However, plastic deformation

under full water load still had to be anticipated.

Separate Contract Work

The replacement of the damaged facing by a temporary timber facing, as shown in Fig. 6, was undertaken by separate contract. This work included the removal of the laminated facing, the repair of the sub-slab and the placing of the timber facing. Also under separate contract additional rock was derrick-placed on the crest of the dam. Both contracts were completed in the Summer of 1935. As may be seen from details a, b, c and d of Fig. 6, the timber facing was assembled from sections, placed horizontally, of three 2'' x 10'' x 10' minimum, random length planks of untreated douglas fir, nailed together to provide tongue and groove interlocks on all four sides, similar to Wakefield sheet piling. Roofing nails along the top groove of each section provided an initial clearance of 1/4 in. in anticipation of additional settlement and strips of galvanized iron were provided to cover open joints at ends of three-plank sections. Walers in vertical planes anchored to the rockfill below served to hold the facing in place and to avoid floatation. It was expected that all but negligible settlements would cease to exist within five years from the time of completion of the timber facing, that the latter would last that long and would then be replaced by a permanent, reinforced concrete facing of more nearly conventional design. Actually, the timber facing lasted more than twice the anticipated period of its lifetime.

Performance Under Full Water Load

An intense storm between February 26 and 28, 1938, which was followed by a major storm between March 2 and March 5, produced the largest flood of record in Los Angeles County. All the flood peaks occurred on March 2.

At Cogswell Dam the peak inflow amounted to nearly 25,000 c.f.s., the reservoir filled in 71 hours and the flow over the spillway peaked at 23,400 c.f.s., although the outlets, with the exception of the lowest two, discharging directly into the main tunnel, remained operative. The lowest two outlets became inoperative after the cage type trash rack collapsed because of clogging by water-logged trash.

The leakage through the timber facing and the dam reached a maximum of 130 c.f.s., whereas, calculations, based on uniform porosity, indicated that leakage for full water load might reach 200 c.f.s. which would still have been less than half the rate of leakage that might prove to be critical. Due to the swelling of the timber facing the leakage was smaller for full than for partial water load.

Damage to Floater Slab

While the timber facing suffered no structural damage under full water load, the floater slab at both abutments developed a crack, extending from the base of the dam to roughly one-half of its height. This was due to bending stresses caused by the afore-mentioned fulcrum effect. The reinforcing steel near the top of the slab was either stressed beyond its elastic limit or had broken the bond by shearing off in form of a spall, the concrete cover. In either case the crack was of such width as to contribute materially to the leakage through the dam. Fig. 7 shows the crack in question near the base of the left abutment.

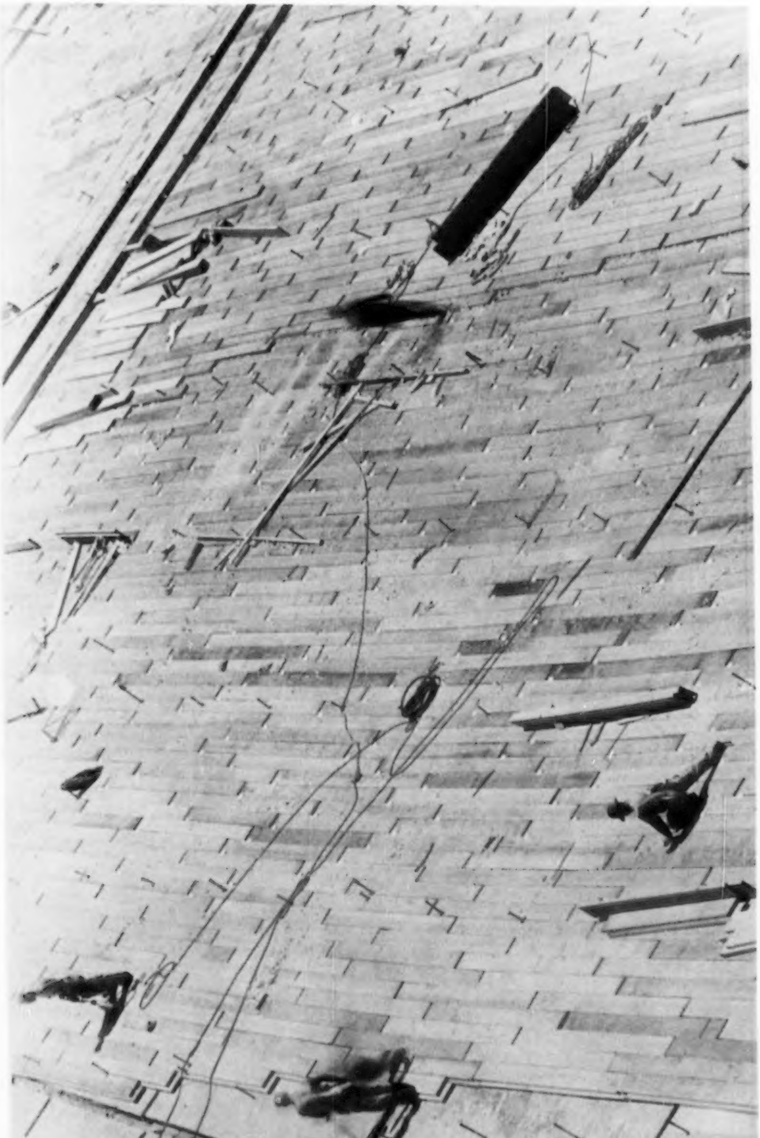
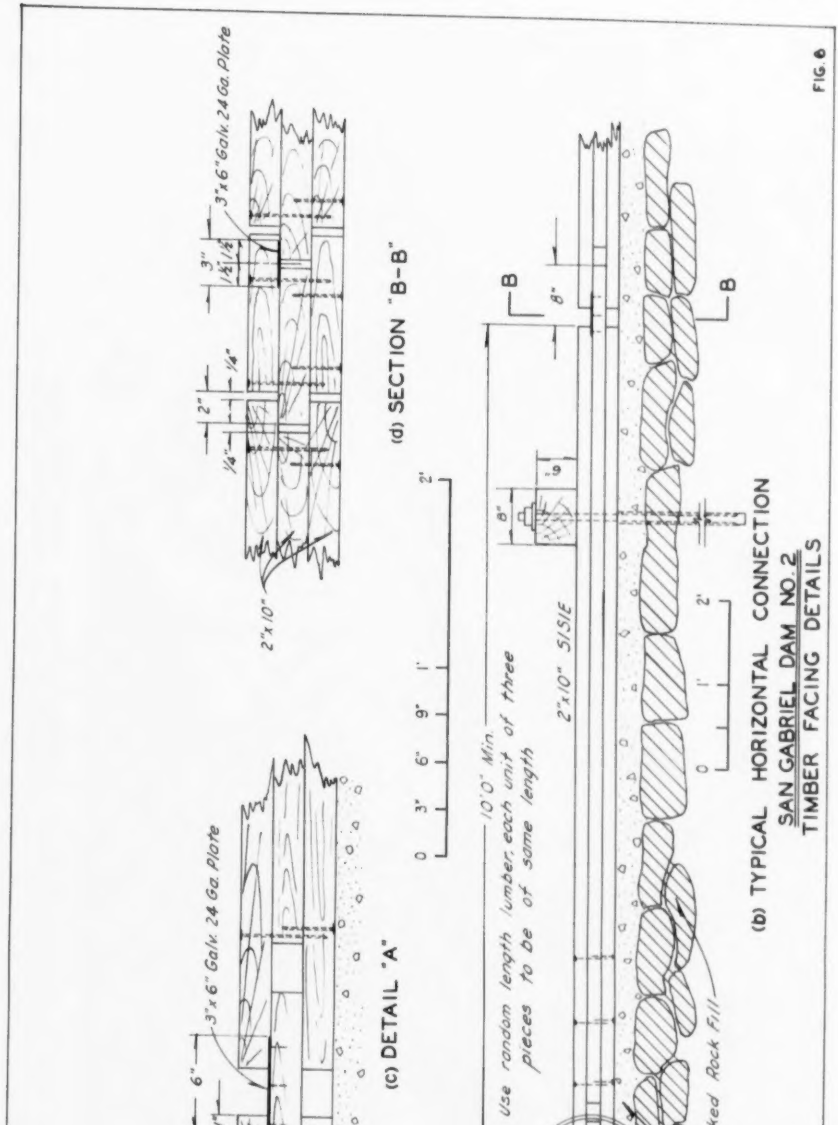


Fig. 6 (a) Placing of Temporary Timber facing over repaired sub-slab of San Gabriel Dam No. 2



(b) TYPICAL HORIZONTAL CONNECTION

SAN GABRIEL DAM NO. 2

TIMBER FACING DETAILS

FIG. 6

Fig. 7 Floater Slab Failure near base of left abutment
San Gabriel Dam No. 2

Revision of Trash Rack

The collapse of the main trash rack called for the change from the cage type to the riser or tower type of trash rack, partial completion of which is shown in Fig. 8. This was accomplished in 1939 by District forces. The reinforced concrete structure was designed to support 340 feet of water column at the base, whereas the trash rack bars were designed for 224 feet of water column, although the design of the trash rack was such as to render clogging extremely improbable. As shown in Fig. 9 the bars were fabricated, tapered box sections with a clear spacing of 15.0 in. on the outside and 15.2 in. on the inside. Hence, any piece of trash clearing the outside spacing will move through without danger of wedging between the bars.

During the flood which culminated on January 23, 1943 the tower type trash rack performed to full satisfaction, although the trash produced was quite comparable in volume with that of the March 2, 1938 flood.

Control of Outlets

The design of the valves for the control of the outlets followed the important requirement for flood control operation of relative invulnerability to clogging by trash having passed between the trash rack bars. Accordingly, as a "rule of thumb," the minimum valve opening should be about twice the maximum trash bar spacing. This rule was satisfied at all outlets including the two 84-inch butterfly valves, protected by the tower trash rack. There is, of course, no harm done if the trash rack spacing is less than one-half of the minimum valve opening, so long as the trash racks are properly designed for water pressure in light of the increasing likelihood of clogging with diminishing bar spacing. However, for reasons of economy in flood operation, it is desirable to have as much trash as possible carried through the outlets by the water and thereby save the expense of manual removal. Obviously, trash such as logs that could cause serious damage to structures or facilities below a flood control dam, should not be permitted to pass through the outlets. Exempt from this restriction naturally is uncontrolled spillway flow.

The control of the outlets at Cogswell Dam, while advantageous for flood operation proved less desirable for operation in connection with water conservation. This was because of the fact that butterfly valves are not stable for part opening, that they should be either fully closed or fully open, and that when fully open the discharge may be far in excess of that desired. To make release of small flows possible, the lowest of the inclined shafts was converted in 1953 by District forces, to an outlet controlled by a 16-inch hollow jet valve.

Permanent Reinforced Concrete Facing

The design of the permanent, reinforced concrete facing, constructed in 1947 under contract, is evident from details shown in Fig. 10. To preserve the integrity of the facing in case of additional settlement, possibly due to a combination of full water load and earthquake forces, special attention was paid to the copper water seal, particularly at junction points of three and four seal sections. This was solved by brazing these sections to a semispheric shell which could stand considerable distortion without rupture. The facing was again articulated in 30' x 30' square units except near the abutments where the shape of the units had to be adapted to the respective

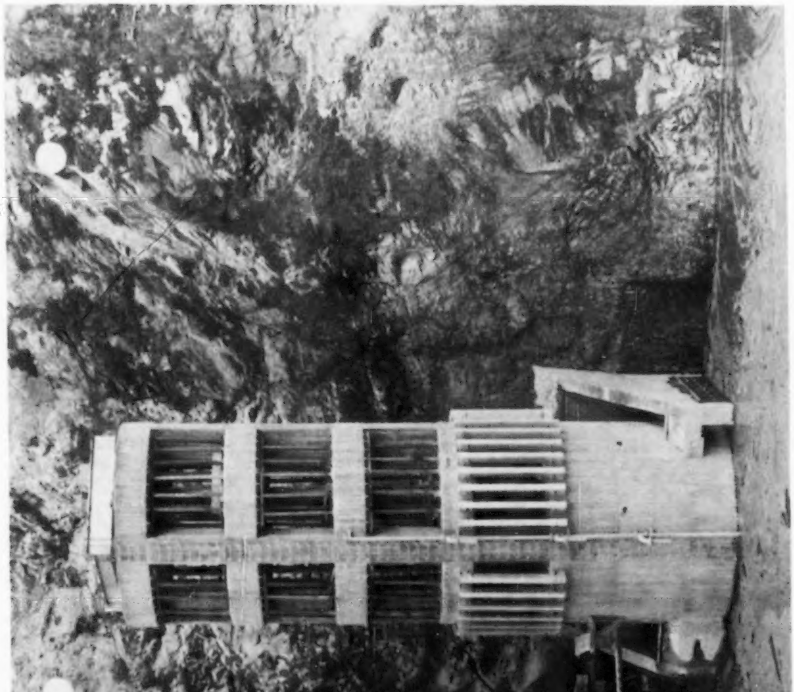
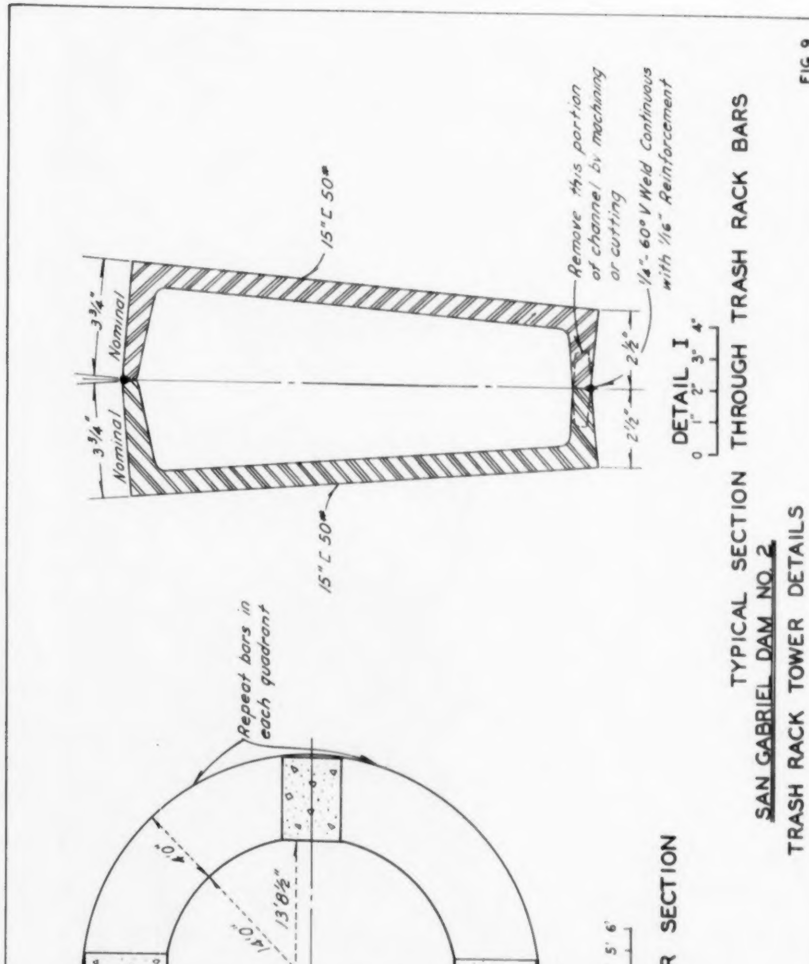


Fig. 8 - Tower Type Trashrack - partially completed.
San Gabriel Dam No. 2

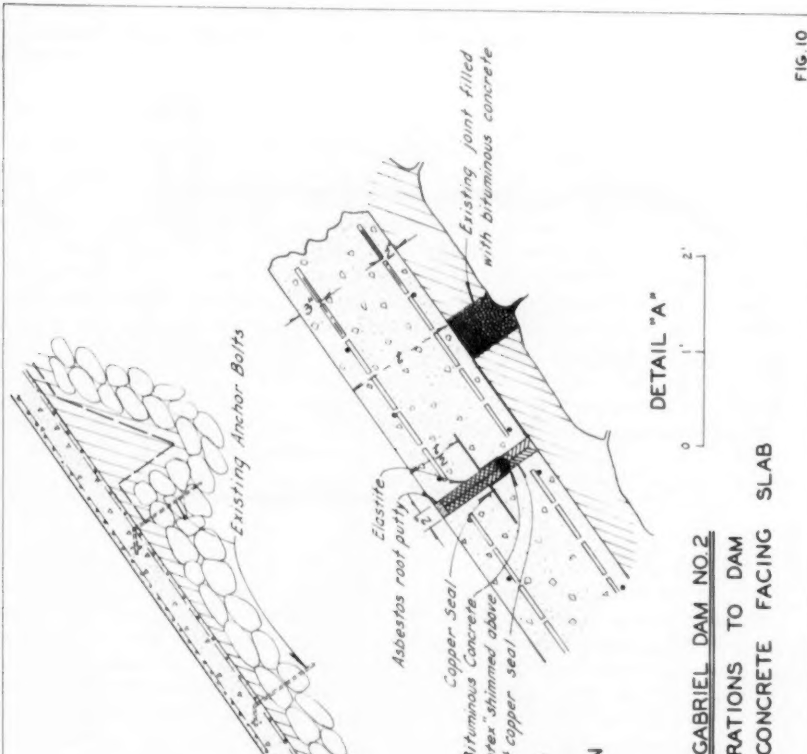


TYPICAL SECTION THROUGH TRASH RACK BARS

SAN GABRIEL DAM NO. 2

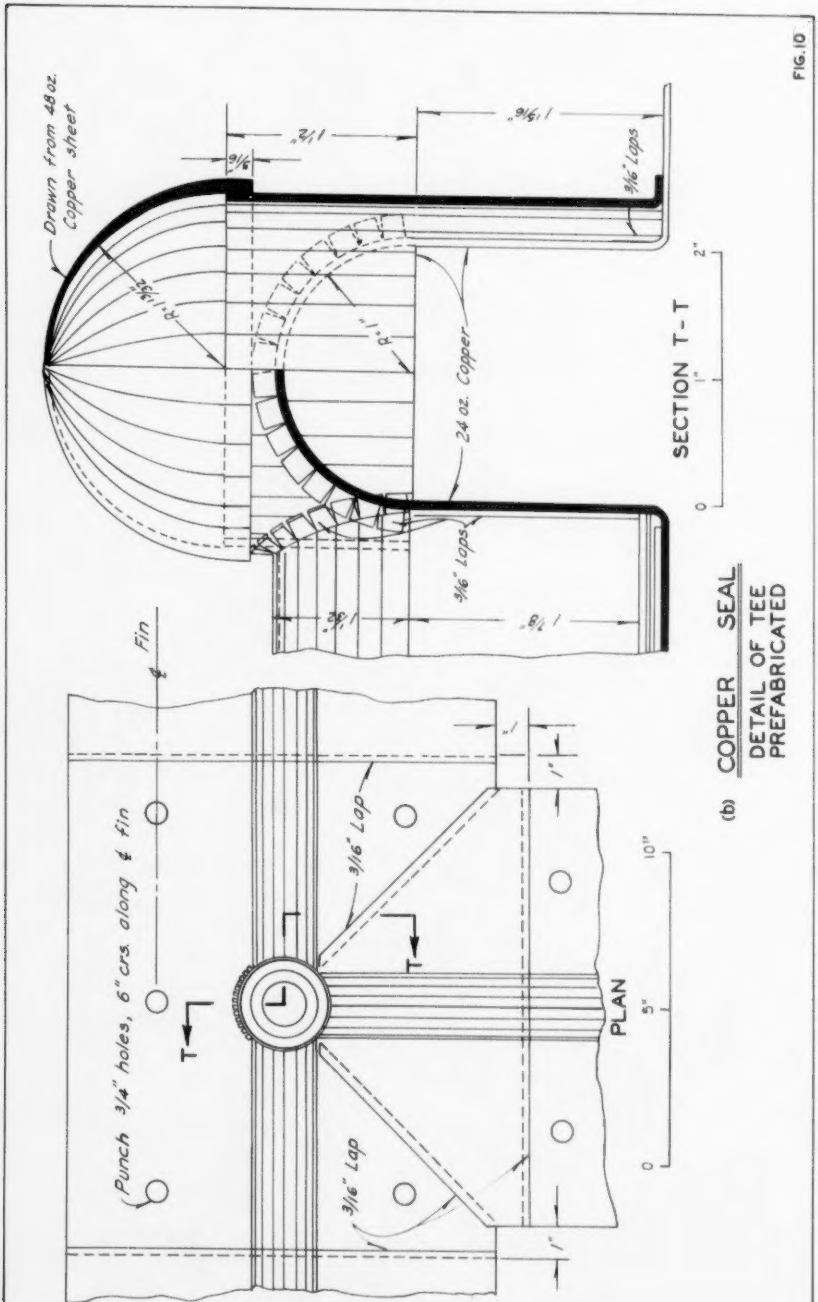
TRASH RACK TOWER DETAILS

FIG. 9



SAN GABRIEL DAM NO.2
ALTERATIONS TO DAM
REINFORCED CONCRETE FACING SLAB

FIG. 10



(b) COPPER SEAL

DETAIL OF TEE PREFABRICATED

FIG. 10

configuration. The reinforced concrete facing was terminated along the 2375 contour and the portion above constructed of broom-grouted rock by District forces in 1948.

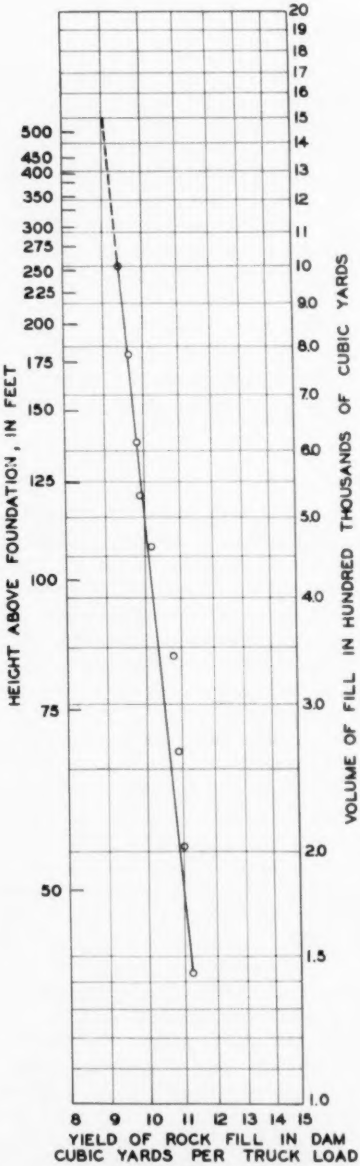
Initial and Residual Settlement

A brief review of the shrinkage and settlement of the rockfill, during and after construction, should prove of interest to the profession and possibly call for caution so far as rockfill dams of major dimensions are concerned. The contractor chose to haul the rock from the quarry to the dam by a fleet of trucks of nominal 10 cu. yds. capacity, capable of carrying 12 cu. yds. The average volume per truck amounted to 11.5 cu. yds. Exact count of trucks delivering rock to the dam was kept and the rockfill measured after placing. Thus, it was possible to derive the relation between truck count and yield in volume of fill in the dam at various elevations. The result is shown in Fig. 11; namely, that the yield of each truck load was 11.3 cu. yds., for the first 25-foot lift and only 9.4 cu. yds. by the time the dam had reached its maximum height. This corresponds to a shrinkage or consolidation during construction of roughly 18 per cent. Residual settlement of 6 per cent previously mentioned, raised this figure to a maximum of 24 per cent and an average of 22.5 per cent.

Significance of Settlement

Loose rock on the truck, weighing 105 p.c.f. (based on 174.7 p.c.f. solid weight and 40 per cent voids exclusive of quarry dust) had consolidated to weigh 128.0 p.c.f. average in the dam during construction and to 135.5 p.c.f. subsequent thereto. Bulging in the lower half of the upstream face, previously mentioned, was too small to have had an appreciable effect on the latter figure. Hence, void space had decreased from 40 to 26.7 and 22.5 per cent., respectively. The shrinkage during construction was primarily due to the crushing of contact points, chips and small rocks with resultant swelling of the crushed material which could only find room in the voids. While this transfer of materials to the voids obviously did not change the total weight, it increased the average unit weight by decreasing the size of the voids. This situation was accentuated through the residual, that is post-construction, settlement which could only take place through additional compression of the crushed material in the voids once point contact had been eliminated. Indeed, the transmission of pressure through the compacted material in the voids is the pre-requisite for final stability of a rockfill as further crushing of contacts is thereby neutralized. To visualize rockfill dams of major height as a mass of large rock with large interstices throughout is therefore a myth.

The permeability of the fill, particularly in the lower part of the dam was necessarily reduced, since consolidation as reflected in above figures is not uniform but varies from a maximum at the base to a minimum at the crest of the dam. Had, for example, the total specific consolidation been close to twice the average at the base and close to zero at the top, the void space at the base capable of transmitting significant percolation would have completely disappeared. On this basis the composite dry density at the base of the dam would be of the order of 156 p.c.f. and the void space 11 per cent. The 105 lbs. of solid material which at 40 per cent voids had occupied one cubic foot of volume prior to consolidation during and after construction would now occupy .67 cu. ft., more or less. It now remains to be ascertained under what



SAN GABRIEL DAM NO. 2

SHRINKAGE OF ROCK FILL AFTER PLACING IN DAM

FIG. 11

conditions the crushed materials, together with the quarry dust could (1) fill the voids, and (2) be compressed to such a density as would render this combination of materials practically impervious.

The weight of the quarry dust which has so far been neglected, must now be added to the weight of the crushed materials. The specifications allowed 3 per cent of the total weight of Class A rock in quarry dust. Hence, quarry dust weighed 3.2 pounds. If in the course of shrinkage during construction 75 per cent of the original 105 lbs. of rock remained uncrushed and 25 per cent of crushed material and quarry dust filled the voids, then the dry density of the combined materials filling the voids would be 118 p.c.f. Thus, in accordance with this premise which seems to be reasonable, the materials filling the void space at the base of the dam were already approaching impermeability prior to post-construction settlement. The corresponding composite dry density, including quarry dust, would be 154.5 p.c.f.

Consolidation due to the residual settlement raised the dry density of crushed materials and quarry dust to 132 p.c.f. or, to at least 95 per cent relative density based on 40,000 ft. lbs of compactive effort. The corresponding composite dry density, including quarry dust, would be 159.2 p.c.f. Hence, even if specific consolidation at the base had been less than twice the average for the entire dam, the rockfill there would still be far from free draining.

Therefore, rockfill dams of major height can by no means be considered immune to pore pressure, especially in the interior of the lower part of the dam where densities would normally be greatest. In brief, this means that a core of low permeability is likely to be formed in such dams whether it be intended or not.

Fig. 12 shows the dam with the permanent facing completed.

Performance of San Gabriel Dam

Original Design

In view of the previously mentioned paper on the "Design and Construction of San Gabriel Dam No. 1" the performance thereof will be confined to those features which are contra-distinct in relation to Cogswell Dam and to those not included in the afore-mentioned paper. Actually the paper refers to the revision of the dam as originally proposed. Basically, the original design was identical with that of Cogswell Dam. However, certain adaptations to the additional hundred feet in height were necessary, among them the number of face laminations and the upstream and downstream slopes. Had the quality of rock, method of placing, and shape of dam site been identical with those at Cogswell Dam the shrinkage during construction as an average for the entire section would, in accordance with Fig. 11, have been 21 per cent and the average total shrinkage 25.5 per cent. Consequently, in view of above statement regarding non uniform distribution of densities at least the lowest 100 feet of the interior of San Gabriel Dam, as originally planned, would have been virtually impervious. Yet, the concept (not necessarily confined to the subject dams) has been that any leakage through the facing would be carried away through the mass of free draining rock and that uplift on the facing in case of rapid draw-down of the reservoir could not occur. With rockfill dams of moderate height and high quality rock relatively free drainage may exist, whereas the assumption that some channels would always be open for drainage,

Fig. 12 Permanent Facing Completed. San Gabriel Dam No. 2

regardless of height, is purely speculative and has no justification in fact. Actually, the rock quarried immediately below San Gabriel Dam (Quarry No. 10) was far inferior in quality to that at Cogswell Dam and proved to be unsuitable for the type of structure proposed. The break-down was such that the fines predominated. It therefore became necessary to revise the design so as to incorporate the fines in the mass of the dam. Fines are defined as particles 1/4 in. in size and smaller.

Revised Design

Figs. 13 and 14 show the revised design of San Gabriel Dam in plan and section. Conspicuous above all, are the flattened faces and the division of the section into six zones. Among the latter zone 3, consisting of quarry rock which had passed through 6'' x 9'' "grizzly" openings, corresponded approximately in cross section, to the original loose rockfill section. The material in this zone, with rock up to 9 in. in size and fines graded down to quarry dust averaging some 60 per cent by volume, was mechanically compacted to a minimum dry density of the fines of 120 p.c.f. Actually, this density averaged 122.2 p.c.f. for zone 3. The composite dry density (rock and fines) averaged 144 p.c.f. Hence, in principle, this quarry material was artificially compacted to a similar density that existed near the base of Cogswell Dam due to self compaction. The zone 3 material was relatively impervious, namely, it showed a percolation rate of 3 feet per year under a gradient of 1 to 1. Had it not been for the earthquake hazard in this part of the country, zone 2, a blanket of moderately compacted clayey sand or loam, would have been superfluous. However, under the circumstances, this blanket, treated so as to remain plastic, was intended to seal possible rupture of zone 3 caused by earthquake.

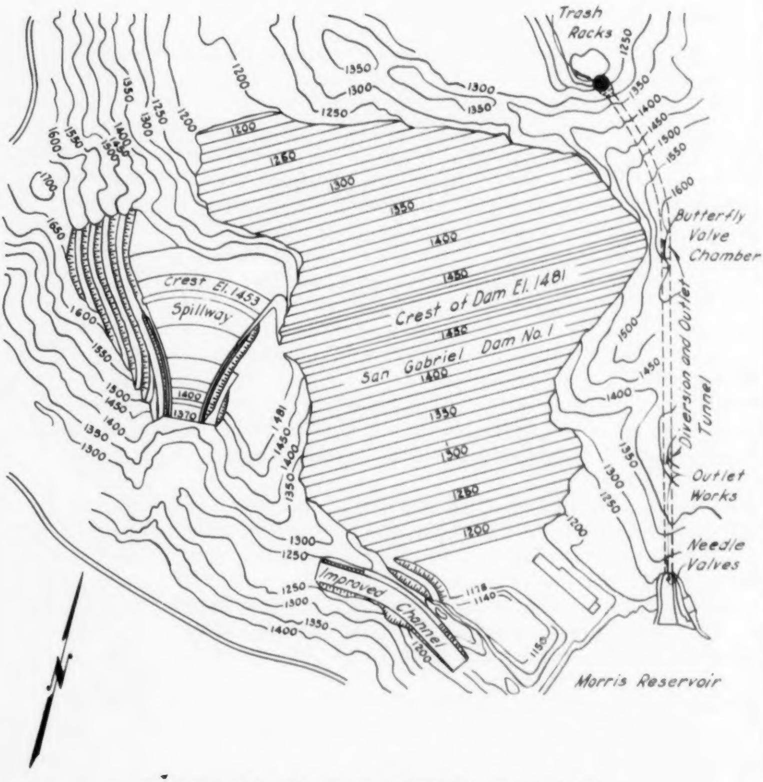
Rock of best available quality was placed in zones 5 and 6, and on the face of zone 1. Otherwise, the rockfill was a mixture of coarse and fine material, roughly graded from finest next to zones 2 and 3 to coarsest toward the face rock of zone 1 and zones 5 and 6.

All loose rock was placed by the use of 2 cu. yds. of water to 1 cu. yd. of rock. Initial settlement, thereby produced averaged 6 per cent. This checks the afore-mentioned settlement at Cogswell Dam due to rain and sluicing after dry placing of the rock.

Great care was exercised in the construction of the cutoff wall which varied in depth between 25 and 80 feet, more or less. Depth of grout holes averaged an additional 73 feet.

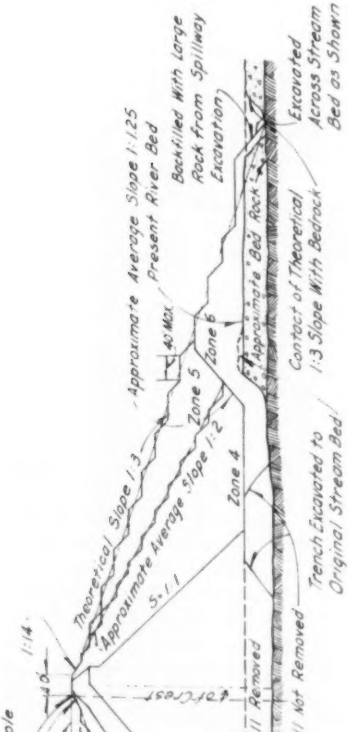
The design of the spillway, placed in a cut through the right or west abutment was similar to that at Cogswell Dam. Its maximum capacity is 290,000 c.f.s. for the controlled and uncontrolled drainage area of 202.7 square miles.

The diversion tunnel, 30 feet in net diameter, served to accommodate the outlet works shown in Fig. 15. Before completion thereof, the aforementioned flood of March 2, 1938 that produced a peak inflow of nearly 100,000 c.f.s. on March 2, 1938 and a spillway discharge peak of 56,700 c.f.s., filled the reservoir in 73 hours. It deposited over ten million cubic yards of debris reaching some 60 feet in depth at the outlet tower. To preserve the effectiveness of the latter, two tiers of trash rack were added in the Summer of 1939. The dam proper was completed in the Summer of 1937, the spillway two weeks before the referenced flood, and the outlet works in 1939. Under the maximum water load the dam deflected downstream 0.11 feet at the crest and returned



PLAN VIEW OF SAN GABRIEL DAM NO. 1

FIG. 13



COMPOSITE MAXIMUM SECTION, SAN GABRIEL DAM NO. 1

FIG. 14

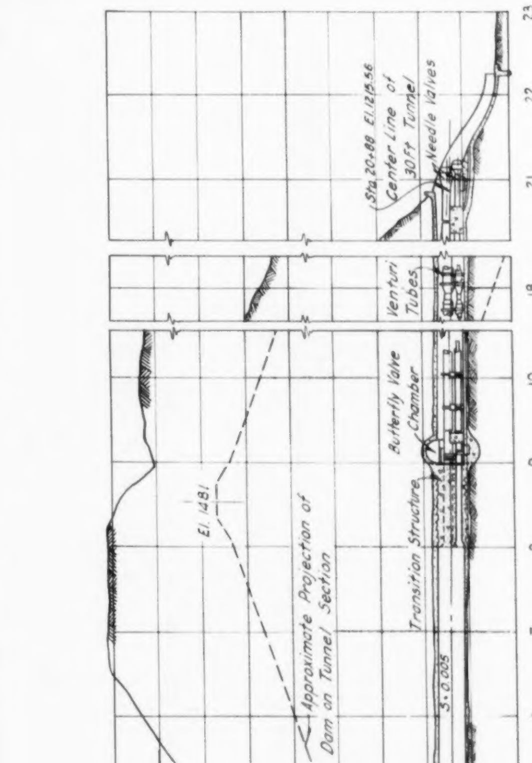


FIG. 15

SAN GABRIEL DAM NO. 1

LONGITUDINAL SECTION ON CENTER LINE OF 30FT. TUNNEL

(elastically) within a fraction of an inch to its position before the flood when the reservoir was emptied. Maximum residual settlement at the crest to date amounts to 0.40 feet, or .106 per cent of the maximum height.

To explore the condition of zone 3 and simultaneously ascertain the position of the phreatic line, a three inch diameter hole was drilled by District forces in 1950 and lined with a perforated casing. The location is indicated in Fig. 14. It was possible to recover cored samples. The average dry density of these samples was 133.4 p.c.f., and the average moisture content 7.6 per cent.

The total residual settlement up to date of the loose rockfill, measured vertically above the downstream toe of zone 3 amounts of 1.57 feet. Fig. 16 shows the completed dam and the spillway under construction.

Epilogue

The design of a loose rockfill dam by far exceeds in simplicity the analysis of its inner stability. In fact, an exact stress analysis is impossible because of the singular absence of homogeneity and isotropy. Hence, this type of dam must be classed as empirical rather than scientific. Yet, the outer and inner stability of a properly designed rockfill dam under static loading cannot be questioned, nor the stability under both static and dynamic loading once the rockfill has fully settled. However, to prevent movement of face rock under dynamic loading, such as caused by earthquake, the slope of the faces should be flatter than that corresponding to the angle of repose. Therefore, it should at least be of the order of 1 on 1-1/2. Even a relatively small differential will, however, enhance safety against movement.

To the writer's knowledge, none of the major, "western type" rockfill dams has so far been exposed to a major earthquake. If so exposed, some settlement in addition to that previously observed would certainly have to be expected, especially in the upper part of the dam least consolidated through self-compaction. It is precisely therein that there rests the difference between a loose rockfill dam and a compacted rockfill or earth-fill dam. In the latter case, homogeneity and isotropy are mechanically created throughout the section and independent of self-compaction. Furthermore, the face slopes of compacted fill dams are invariably a multiple of the slope of repose. Where there is a choice and costs compare favorably, compacted fill dams should be given preference, especially in areas known to be subject to earthquakes.

ACKNOWLEDGMENT

The writer is indebted to Mr. H. E. Hedger, Chief Engineer of Los Angeles County Flood Control District, for his support in the preparation of this paper. To Messrs. R. D. Reeve, E. C. Kenyon, Jr. and C. J. Reinhard and Mrs. Mary K. Imandt for assembling and drawing the exhibits. Also to Mrs. Hazel M. Robertson for recording, typing and assembling the paper.

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Fig. 16 San Gabriel Dam No. 1 completed. Spillway under construction

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CONTENTS

DISCUSSION
(Proc. Paper 1689)

DESIGN OF LARGE PRESSURE CONDUITS IN ROCK[a]

Discussions by D. J. Bleifuss, F. L. Lawton, Pierre E. Danel and Jean Rueff

D. J. BLEIFUSS,[1] M. ASCE.—The writer has read the paper by Messrs. Patterson, Clinch and McCaig with deep interest, and considers it a valuable addition to the literature on large pressure conduit design.

In arriving at the proportion of stress carried by a steel liner, the authors have considered (a) the deformation of concrete, (b) the deformation of the rock, (c) the probable gap between steel and concrete caused by cooling of the steel when the penstock is filled. To the sum of these deformations they equate the deformation of the steel. The resulting distribution of stress between steel and rock is a very good guide to judgment, and we as yet have nothing better, but it must be borne in mind that it cannot possibly be an exact solution. It would be interesting to know if any field observations of stress distribution were made on the cited installations, and, if so, what the results were.

Since the design of a pressure conduit includes specifying the concrete mix and the placing thereof around the steel liner, and grouting (if deemed necessary), it appears these merit careful consideration. The idea is to fill completely with concrete all space between the steel and the rock. Placing concrete will not accomplish this, so it is customarily supplemented by grouting, working from inside the liner.

This grouting operation is always an unsatisfactory affair for several reasons. First, it is expensive. Second, the holes through the steel liner are a nuisance, both to provide and to close satisfactorily. Third, it has to be done very carefully, or the steel liner will be bulged in places. Fourth, it probably results in uneven external pressure on the liner. But, overshadowing all the others is the fifth reason; this grouting is necessarily a blind operation. The grout holes are ordinarily provided in predetermined locations, which of course can bear no relation to places where the grout is most needed. When grouting is completed, one cannot know what one has accomplished, one can only be certain he has not accomplished everything he intended.

It has long seemed to the writer that if a method could be devised for completely filling the space between steel and rock, at the same time leaving some initial compression in the liner, it would be highly desirable. This leads to consideration of an expanding concrete which would do both. It would seem that its placing would have no more uncertainties than grouting, and that grouting might be avoided.

a. Proc. Paper 1457, December, 1957, by F. W. Patterson, R. L. Clinch, and I. W. McCaig.

1. Cons. Engr., Atherton and Sacramento, Calif.

When a steel-lined conduit is kept under pressure for a considerable time, the concrete deforms. Part of the deformation is elastic, and part of it is plastic. When the pressure is removed, the concrete thus does not return to its original position, but the steel will return to its original dimensions. A gap must therefore form between steel and concrete. There is probably a gap there already, due to the cooling on the first filling, and will probably be increased. This is regardless of the fact that it has been grouted very carefully.

Unavoidably, in a tunnel, there will be spaces at the crown which will not be filled with concrete. These must be grouted. But it would seem that grouting for any other purpose is pointless.

F. L. Lawton,[1] M. ASCE.—The authors are to be congratulated on an excellent paper, which constitutes a distinct addition to the literature on the design and fabrication of large pressure conduits in rock. The authors' explanation of the rock stresses around cavities is most lucid. It is to be regretted they made no reference to the basic work of Heim.

It is unfortunate the authors' treatment of rock stresses due to water pressure in a conduit should have led them, no doubt inadvertently, to make such a statement as "Under such ideal conditions, the factor of safety against tension stress occurring will be p/2wz and, if w is taken as 156 pounds per cubic foot, the cover can be as low as 20 per cent of the head in the tunnel before the safety factor is reduced to unity". Consideration will show, it is suggested, that the foregoing should more correctly read "Under the conditions illustrated by Fig. 2(A), where N equals 1, and Fig. 3, the factor of safety against tension stress occurring will be 2wz/p and, if w is taken as 156 pounds per cubic foot, the cover can be as low as 20 per cent of the head of water in the tunnel, expressed in feet, before the safety factor is reduced to unity".

The authors' treatment of design requirements to resist external water pressure is excellent and their Fig: 4 constitutes a major contribution to published criteria on the subject. It should be emphasized that ". . . the maximum external pressure head that can exist outside the steel lining will not exceed the depth of cover over the steel lining".

The authors are to be particularly congratulated on their observations relative to brittle failure of steel, and the importance of proper attention to this factor in steel liners of pressure conduits subjected to temperatures in the order of 40° F. or lower. The extent to which the engineering profession has failed to recognize the significance of brittle failure at low temperatures is surprising.

Fig. 4 indicates that buckling of the Bersimis No. 1 pressure conduit steel occurred at external pressures of approximately 45 psi, 100 psi and 300 psi. It would be useful if the authors described the nature and extent of the buckling, particularly as the 3A failure indicates, as does Fig. 4, that grouting was carried out in three stages whereas the text indicates only two stages were employed, low-pressure grouting at 100 psi and high-pressure grouting at 300 psi for penstocks No. 1 and No. 2, with only the low-pressure grouting for the others.

Although not stated it is implied the Bersimis No. 1 penstocks and the supply tunnel were filled by pumping rather than admission of water from the intake. Could the authors clarify this?

1. Chief Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

PIERRE E. DANEL,[1] M. ASCE and JEAN RUEFF.[2]—The very interesting paper by Messrs. Patterson, Clinch and McCaig will call for a few comments from the writers concerning the action of external pressure.

1. Studies and test concerning the collapse of thin shell liners under external pressure were carried out by Sogreah and Sorefame. The first studies are dated 1944.

   The test models were 8" and 4'6" in diameter, with thickness to radius ratios varying from 1% to 0.56%. The large diameter models were constructed according to the usual practice in penstock and steel lining construction. These liners were embedded in sandless, low strength concrete contained in a thick external sheath. Both ends of the sheath were fitted with special flanges designed to provide watertightness without any constraint on liner ends. The shells were not grouted and when hammering their internal surface, one could note hollow echos on surfaces of a few inches to a full square foot.

2. A few tests were carried out on unstiffened steel liners. These tests showed that the failure would occur at pressures somewhat greater than those computed from Amstutz 1953 formulae. (Note that the diagram on Fig. 4 by the authors is not in agreement with these formulae).

   The differences between experimental results and Amstutz theoretical computations can be easily explained by the fact that Amstutz did not take into sufficient account the continuity of the shell when considering the part in contact with the surrounding concrete and the part not in contact.

   The stresses in the steel linings can be computed with base on differentials Eqs. 12, 13 and preceeding ones of the article by Professor Anglès d'Auriac in "La Houille Blanche" no. 3 May/June 1947, with no more assumption than the continuity of the lining.

3. It is European practice to reinforce linings against external pressure by circular rings welded to the external surface of the shell.

   An intensive series of tests on this subject has been carried out by Sorefame on 4'6" liners, 15' long, for thickness to radius ratios from .73 to .56% and for various reinforcements. New tests will proceed for larger thickness to radius ratios. The influence of both ring characteristics and spacement is investigated.

   Although one could expect a somewhat large dispersion in test results, practically no dispersion was observed when measuring the buckling pressures. As a matter of fact the buckling of all the various sections of a reinforced test model always occurred for the same pressure notwithstanding the small imperfections of the liner, constructed according to the usual practice and not annealed. One could only note difference in time between buckling of one section and of another.

   Buckling of the stiffening rings always followed the buckling of the shell, for only a very slightly higher pressure.

   The experimental results are in good accordance with data from field failures of liners during grouting, when available.

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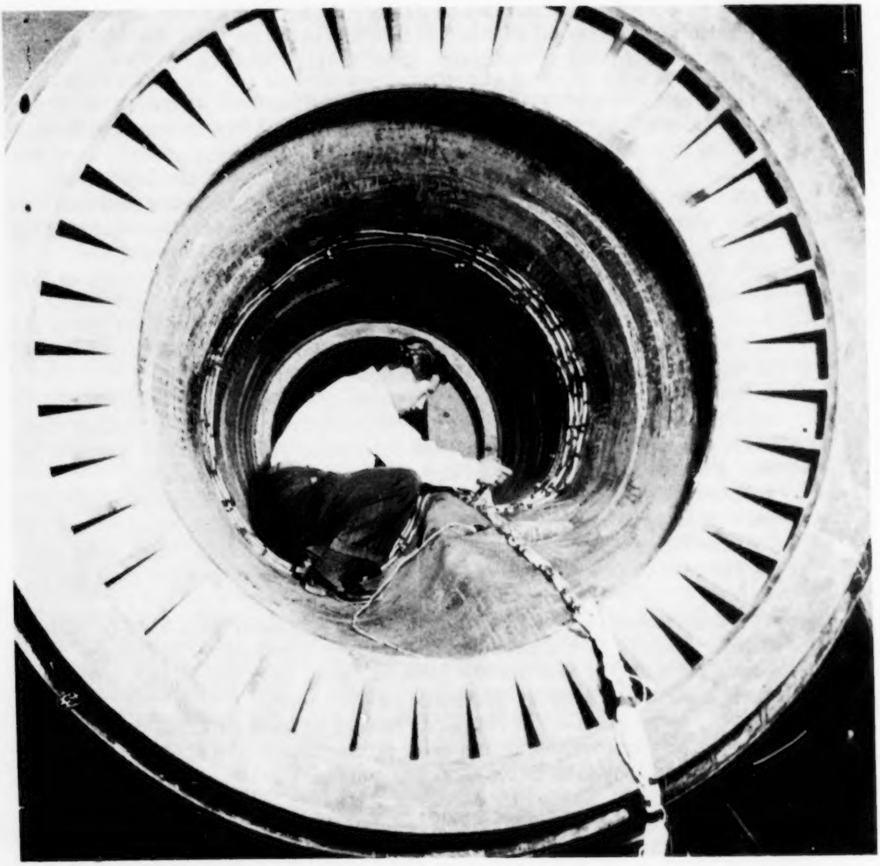
1. Pres., Sogreah, Grenoble, France.
2. Chief Design Engr., Neyrpic-Sorefame, Amadora, Portugal.



4. To show the interest of stiffening rings, we may compare Bersimis 2 liners (17' I.D., 3/4" thick, no stiffening rings) with Picote liners—Douro river, Portugal—(18' 17/32" I.D., 25/32" design thickness, stiffening rings made of 8" x 4" x 5/8" angles every 4'4"1/2).

Both linings have practically the same diameter to thickness ratio—272 for Bersimis, 275 for Picote. The first lining could withstand an external pressure of 75 psi according to Fig. 4 with an initial gap 3.10^{-4} of radius, 68 psi according to Amstutz 1953 formulae, with no initial gap. The same lining in our test conditions could withstand 71 psi approximately. Buckling of the Picote liner in 1 to 4 scale model occurred only at 216 psi, although it was a little more slender than Bersimis 2. The failure of a stiffening ring, torn out of the concrete, was observed when the external pressure reached 227 psi. Under this pressure, the compressive stress in the shell, computed without taking buckling into account is 31,200 psi, approximately equal to yield stress.

By comparing these results we conclude that with an increase of 17, 2% over the weight of the bare shell, the liner can withstand an external pressure 3 times greater.

Note also that the erection and concreting of a ring stiffened liner is much easier than that of an unstiffened one, as no special provision is usually required to avoid large deflexions.

5. The choice of the design value for the external pressure acting on a lining is usually uncertain. Even if precise field measurements are carried out, there is no guarantee that the observed pressure is the maximum one. As a matter of fact, the pressure is a function of the hydro-geological structure near the site and of the hydrology of the period immediately preceeding measurements. In certain cases the effective external pressure may exceed the rock cover and/or the static head.
6. Even if a lining is designed after precise computations and extensive model tests and in particularily simple hydrological conditions, it seems fitting to use a safety factor in the same manner as one does in the classical case of pressure piping or any other construction job. It has been our standard practice to apply this factor to the design pressure. Phenomena involved not following linear laws, this is not equivalent to applying it to stresses. Common values of the safety factor have been 1.5 or 1.6.
7. An article on the present tests is under preparation.

CIRCULATING WATER SYSTEMS OF STEAM POWER PLANTS[a]

Discussion by D. I. H. Barr

D. I. H. BARR,[1] A. M. ASCE.—The author restricts himself to the design of the most efficient conduit system to move water between predetermined points of intake and discharge. But he might, with advantage, have mentioned that such a process cannot often be entirely separated from the general optimisation of the circulating water and condensing arrangements.

For a particular site, representative natural water temperatures for portions of the year should be selected and the values of incremental output throughout the year compared with capital and running costs for a range of condenser temperature increments and allowing for recirculation, if applicable, appropriate to the various possible layouts. Statutory limitation on total temperature use, comprising condenser, and recirculation increments may also have to be considered as may different load factors at the various representative natural water temperatures. Thereafter, the effects of a selection of the more promising alternatives should be used in general plant optimisation.

Several papers on plant optimisation and related problems have appeared recently in the Transactions of the American Society of Mechanical Engineers.(1,2,3,4) Methods of systemising optimisation are given, though at the risk of vitiating the process by the effect of too many simplifying assumptions; this applies particularly to the civil engineering aspects and especially to heat dissipation. But it is clearly shown that sufficient choice is available on the considerably more standardised mechanical side to make component selection by no means cut and dried. The figure of 300 cfs. for a 200,000 KW unit, as given in the paper, represents a condenser increment of the order of 14° F. But the range in design vacuum (rated back pressure) quoted in the Power Magazine installation lists for large capacity plant demonstrates that, allowing for variations in the selected design intake water-temperature, some considerable variation in condenser increment is economically possible. The marginal cost of variation in condensing water capacity can be quite high; of the order of $0.25 per KW per degree change in condenser increment at about 15° F. increment and this makes careful comparison with the capitalised value of change in output well worthwhile. In short, it is perhaps rather easier than is generally recognized by the civil engineer, to design the most efficient conduit system to move the wrong quantity of water between incorrectly predetermined points of intake and discharge.

a. Proc. Paper 1488, December, 1957, by R. T. Richards.

1. Research Student, Royal College of Science and Technology, Glasgow, Scotland.

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THE HAAS HYDROELECTRIC POWER PROJECT[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author has provided a most comprehensive and interesting treatment of the first large underground hydroelectric plant in the United States—the Haas power project—and the associated elements of the development, notably Wishon Dam, the outlet and power intake, etc.

It is noted that the steel trash rack at the intake tower is galvanized. This appears to be somewhat unusual in North American practice, and it would be useful if the author could indicate the relative life as compared with ordinary steel provided with more customary forms of underwater corrosion protection. What is the pH of the water from the Wishon reservoir?

It is noted the design of the intake tower, Fig. 3, provides for the oil cylinders which operate the two 4 ft. x 5 ft. slide gates to be located approximately 30 ft. below the full reservoir level. Has such an arrangement proven to be free from trouble due to corrosion?

The extensive experience of the author's organization with butterfly valves, no less than 95 in sizes from 3 to 13 ft. diameter, is a very strong recommendation for this type of valve.

The comprehensive experience of P. G. & E. with unlined tunnels, where rock conditions permit, totaling no less than 800 mile-years of service, adds significance to the statement "Rock falls in early tunnels were due to not plugging soft seams. Water would wash out the seams and cause large blocks of rock to fall in the tunnel. No rock falls have occurred in tunnels constructed since 1947 where proper dental work has been done during construction". The reference to proper dental work is a recognition of the fact that relatively unsuitable rock can be fully utilized, in short sections, without the necessity of costly lining provided it is correctly treated. Put another way, a full knowledge of rock mechanics is essential to secure the best results from structures involved in underground power developments, not only including pressure tunnels.

The author's statement of P. G. & E. experience and evaluation of the significance of an unlined invert is particularly valuable; it indicates the importance which should be attached to a proper evaluation of the economic significance of lining, whether the invert only or the entire periphery of the tunnel. It would be of interest if the author could indicate if the experience of his organization embraces determinations of Manning's "n" for unlined tunnels with and without a paved invert.

a. Proc. Paper 1529, February, 1958, by J. Barry Cooke.

1. Chief Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

It appears the author's organization has found that ". . . surface muck, to about 2 in. dimension, moves down the tunnel invert during the first year of operation. The larger sized muck then retains the remaining small material and there is little sand or gravel that moves after the first year." Is this material only that arising from the scouring action of a 6 fps tunnel velocity on the material left in the invert?

The author noted that ". . . cracks, several feet long, occurred in the 4-inch plate adjacent to the weld and several inches deep" in the Haas wye and, by implication, inferred this was due to a very tight schedule and the actual welding program. It is suggested the value of the paper would be materially enhanced if a sketch or other illustration indicating the location of the cracks could be included in the author's closure as it appears cracks of one type or another have occurred in many large wyes fabricated in North America.

The thorough-going nature of the economic studies which led to the decision to construct the Haas plant as an underground scheme is thoroughly demonstrated by the author's realistic statement of the economics. The generally excellent granite formation obviously played a very substantial part in the net direct capital saving of about $400,000 shown for the underground scheme. In this connection the author's observation that ". . . the cost and possible contingency in the excavations was a major factor of uncertainty in an underground power plant. The main criterion in selecting a layout was to have minimum excavation" is particularly significant. It should be repeated every time this type of plant is under consideration. Further, excavation in such a manner as to minimize damage to rock intended to serve a structural purpose can result in material savings, in eliminating the dental work otherwise necessitated.

The rock-bolted and gunited arch for the powerhouse is in line with practice adopted in some cases in Sweden. The rather elaborate method of applying the Haas rock anchors—the Perfo method—would appear more expensive than the use of the split rock anchor and wedge where the friction developed between the split end and the rock periphery of the hole is depended upon to develop the strength of the anchor. However, there may have been other reasons for this particular application and it would be of interest if the author could touch on this point.

UNDERGROUND POWERHOUSES IN ITALY AND OTHER COUNTRIES[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author has strikingly demonstrated the basic soundness of the underground hydroelectric power plant in the many applications to date in Italy—no less than 60 already built or under construction since the first at Coghinas, Sardinia, built in 1926.

The variations in the author's designs show very clearly the adaptability of such stations to geological conditions which in many cases were far from good.

The author has drawn attention to the fact that the emplacement underground of a powerhouse and all or part of the head and tail structures gives the designer a certain freedom from the limitations imposed by topographic conditions and often permits obtaining better and more economic solutions. This is particularly significant in respect of the geological conditions, as noted relative to Juquiá No. 1, Bitto No. 4 and Santa Giustina, where the power tunnel was excavated through limestone while the penstock, powerhouse and tailrace were located in marls.

Mr. Marcello has drawn attention to Gerola (Bitto No. 2) power development which, while built with a reinforced concrete structure, was covered with earth to protect it from avalanches. This is a particular condition which may arise in mountainous country.

The author has indicated that the slope generally adopted for the penstocks runs from about 70 to 85%. This appears to be rather steep. Was it done to secure a shorter penstock at the expense of a somewhat longer tailrace or because such a slope is necessary for the economic removal of muck?

The closing paragraph of the paper cannot be too strongly stressed. Full appreciation of the rock mechanics involved in the proposed orientation of an underground power plant normally results in substantial economies and eliminates the likelihood of unpleasant surprises in the behaviour of the rock.

a. Proc. Paper 1554, February, 1958, by Claudio Marcello.

1. Chief Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

THE SUDAGI UNDERGROUND POWER PLANT, JAPAN[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author's informative paper on the first large-scale underground power plant in Japan, the Sudagai plant, is significant for a number of reasons, among others that comparative studies showed that the head-type underground plant was superior on both economic and technical grounds.

The author's reference to the relative intensity of seismic forces at the surface and underground is important. It has long been suspected that greater freedom from earthquake effects might be an important factor in favour of the underground station in certain areas. Accordingly, it is hoped the results of the investigations being made by the Tokyo Electric Power Co. on surface and underground seismic forces will be published in due course.

It is believed the Sudagai plant is the first underground plant to be emplaced with a surrounding drainage tunnel, with upward-drilled vertical drain holes from it constituting a curtain. It is noted the filling of the reservoir apparently did not cause any increase in leakage through the drainage tunnel, which continued at 7 to 8 liters per minute. This seems to be quite a remarkable result in view of the proximity of the power plant to the reservoir. No doubt the coarse-grained granite minimized the passage of water through the rock towards the curtain of drain holes. It would be interesting to know if the author would use the same drainage tunnel technique if he were planning Sudagai again.

Presumably the curtain of drainage holes was drilled after the grout curtain had been completed. In this connection it would be interesting to know what kind of a pattern was adopted for the grouting. To what level was the grout curtain carried out? What was the grout consumption?

The properties of the coarse-grained granite from the power-plant chamber are interesting. The modulus of elasticity and Poisson's ratio were not given. Perhaps the author has such additional information which could be added in his closure.

It is noted rock bolting was employed extensively. Was any provision against corrosion of the rock bolts made, in view of the drainage curtain? What was the pH value for the drainage water?

Attention is drawn in the paper to the construction of a partition wall between the tailrace tunnel and the surge chamber, to reduce the hammering action occurring when air was released from the tunnel into the surge chamber. What was the size of the air pipe installed through the partition wall?

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a. Proc. Paper 1555, February, 1958, by Tatsuo Mizukoshi.

1. Chief Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

The paper observes that ". . . we can confidently affirm . . . that if geological conditions had been more adequate, (a) simplified structure would have been possible with resultant reduction in construction cost". Does this mean the underground development proved to be more costly than the alternative surface plant?

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268 (PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277 (HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286 (PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

c. Discussion of several papers, grouped by divisions.